MEASUREMENT AND ESTIMATION OF SHAFT-RESISTANCE OF PILES

A Thesis Submitted
in Partial Fulfilment of the Requirements
for the Degree of
MASTER OF TECHNOLOGY

by

I. V. ANIRUDHAN

to the

DEPARTMENT OF CIVIL ENGINEERING
INDIAN INSTITUTE OF TECHNOLOGY, KANPUR

APRIL, 1982

dedicated to

grand father

CERTIFICATE

It is certified that this work, 'Measurement and Estimation of Shaft-Resistance of Piles' by I.V. Anirudhan has been carried out under my supervision and that this work has not been submitted elsewhere for a degree.

M.R. Madhav
31.3.82
M.R. Madhav
Professor
Department of Civil Engineering
I.I.T. Kanpur-16.

ACKNOWLEDGEMENTS

It is a great pleasure to thank several people who have been responsible, in various ways, for the completion of this thesis.

It was a life time experience to have been associated as a student with Professor M.R. Madhav. He was much more than a guide in my work. His criticisms, suggestions, help and encouragement have gone a long way in making my thesis see daylight.

The success of this work has been critically dependent on Mr. N.B.S. Rao's suggestions and help in the laboratory at times when they were most needed. My earnest thanks are due to him.

I thank Mr. Lakshmidhar and Mr. A.K. Srivastava for their help in the laboratory.

Mr. S.K. Gupta's assistance in the experiments was a great help. I also thank Mr. Verma for his help in making the drawings. I must not fail to acknowledge the various help rendered by Mr. Trivedi, Mr. Gulab Chand, Mr. Dube and Mr. Parashuram.

The many friendly chats I had with Mr. Abraham helped enliven the work by breaking its monotony. It is impossible to list the many different ways in which Mr. Abraham has helped me. I am greatly indebted to him.

I acknowledge my deepfelt gratitude to the invaluable guidance and encouragement given me by my parents.

Last but not least is the role played by my friends. My heartfelt thanks are due to all of them, who have helped me in various ways.

ANIRUDHAN

Table of Contents

LIST OF FIGURES

SYNOPSIS

If the pile is not instrumented separate measurements of shaft and base loads are not possible. Reasonable estimation of shaft resistance and base resistance separately from an ordinary load test will be useful as instrumentation of piles are expensive. Model pile tests are useful to evaluate the shaft resistance of piles in terms of lateral pressure at the pile-soil interface.

A series of tests have been carried out on model piles in different samples of soft clays and calcarean sands. The practical relevance of this study is that piles are mostly needed in off-shore structures, where sands or soft clay are encountered.

A triaxial cell has been modified and used to conduct the experiments. The samples were prepared with a pile inside and the pile is axially load-tested. The bottom end of the pile was free to move so that no end resistance was offered.

The calcareous sands offered a lower frictional resistance compared to other sands. The effects of strain rate on shaft resistance of piles in clays were studied by a sufficient number of experiments. Rough piles in sand showed a significant increase in shaft resistance.

The inverse slope method (Chin, 1972-'77) was used to analyse Whitaker's pile load test data and was found to be unreliable for those data. A new empirical procedure to estimate shaft resistance of piles is proposed using elastic solutions provided by Poulos and Davis (1980).

CHAPTER 1

INTRODUCTION

Any foundation should serve the quality of transmitting loads or forces to the ground without excessive settlement. It may either be a structure which directly bears the load or a pile foundation. Direct bearing foundations or shallow foundations are constructed as a spread, under the structures to transmit the loads to the soil. When the soil is weak or compressible, and of considerable thickness, piled foundations are necessary to carry the load to an underlying hard or stiff stratum through the weak layer. In the case of soft clay or peat, which are expensive to remove or to excavate piled foundations are the proper solution. Piles may be required to carry uplift loads when used to support tall structures which are subject to overturning forces.

When the piles are being used in deep beds of clay or other medium stiff soils, it is mainly supported by adhesion or frictional action of the soil on the shaft surface. These piles are termed as friction piles or floating piles. When a pile is taken to a hard underlying stratum, most of the load is taken by point-bearing; hence the term end or point-bearing piles. In most of the cases, a pile driven into the soil transmits the total load both by base and shaft resistances in various proportions.

Many research programmes, particularly during the last 25 years, have provided foundation engineers with much of the information they need to design and construct individual or a group of piles. Dynamic formulae, and static formulae based on total and effective stress principles, were developed to predetermine the load carrying capacity of piles, or to estimate the ultimate capacity of the piles driven into the ground .

Research was carried out to study the behaviour of a pile under any type of load and to furnish the most reasonable design methods. Large number of instrumented piles were load-tested to find a rational way of analysis. Model pile tests were proposed to provide a substitute for the field-load-tests.

In Chapter 2 a brief review of the work on prediction and measurement of the pile capacity is given. Mainly the work on axially loaded piles is considered.

The model tests conducted by others in the field are reviewed in Chapter 3. The details of the model pile tests conducted in the present study are also presented in the same chapter. Tests were conducted with a view to identify the factors affecting the frictional capacity of the piles. Two types of calcareous sands classified as sand A and B were used in the present study. The present work included the

study of two clays, Clay C and C in which model piles were test loaded. The physical properties and strength characteristics of the soils also are described in Chapter 3.

Since the purpose of the test programme was to relate the pile capacity to the pile movement and it was found that this could be done best by using constant rate penetration test, of the two types of pile load testing, viz., Maintained Load (ML) test and Constant Rate of Penetration (CRP) test, (Coyle and Reese, 1966), the latter was adopted.

In the analysis of the pile load test data, the main idea is to study the shaft resistance and base resistance individually so that a better prediction of total load carrying capacity is possible. So, in the model tests conducted a purely frictional pile was considered. This was possible by leaving a free space below the pile for the movement of the same.

The methods suggested earlier to separate total load into shaft resistance and base resistance are reviewed in Chapter 4. Pile load test data in London clay furnished by Whitaker and Cooke (1966) are analysed in the same chapter. An attempt was made to analyse Whitaker's date using the hyperbolic stress-strain relation to separate total ultimate resistance into shaft and base resistances, as suggested by Chin (1972,1977).

A semi-empirical procedure to estimate the shaft resistance out of a total load-settlement data is proposed. It consists in the use of elastic solutions for pile foundations provided by Poulos and Davis to calculate the modulus of subgrade reaction at the base of the pile and an estimation of the movement of the pile by which the full shaft resistance is mobilised. A movement of 0.367 percent of shaft-diameter (average) is found to be adequate for the mobilisation of ultimate shaft resistance for the piles in London Clay. This value of 0.367 percent is valid for the piles with enlarged base also.

References are given at the end of the thesis.

CHAPTER 2

LITERATURE REVIEW

2.1 INTRODUCTION

Literature on prediction and estimation of the load carrying capacity of a pile is very extensive. Considerable work has been done to predict the pile capacity based on the soil properties, which resulted in a number of 'static' formulae, called so in order to distinguish them from the 'dynamic' formulae.

The analytical problem presented by a pile installed in real soil and then subjected to loading is one of the extreme complexity. Experiments on axial load capacity of piles are reviewed here and static formulae are given more emphasis than dynamic formulae.

2.2 DYNAMIC FORMULAE

All dynamic formulae owe their existence to the assumption that the resistance for driving the pile is equal to the ultimate bearing capacity of the pile under static loading. Most of the driving formulae (dynamic formulae) are based on energy concept. Empirical formulae have been proposed based on data collected from pile driving (Whitaker, 1970).

A dynamic relation based on wave equation was proposed by Smith (1962) and many have improved on this method.

However, Tomlinson (1977) is of the firm opinion that there is no place for dynamic formulae in the calculation of the ultimate resistance of piles.

2.3 STATIC FORMULAE FOR AXIAL LOAD CAPACITY

Static formulae for predicting the ultimate resistance of a pile are based on the properties of the soil. The basic equation for the pile capacity in the 'static' calculations is

$$Q_u = Q_b + Q_s - W_p \tag{2.1}$$

where

Q_u - ultimate total resistance

Q_b - ultimate base resistance

Q_s - ultimate shaft resistance

W_p - weight of the pile

As W_p is negligible compared to Q_u the equation (2.1) is modified to

$$Q_u = Q_b + Q_s \tag{2.2}$$

As the behaviour of the loaded piles in cohesive soils and cohesionless soils are different, and most of the work was done either on cohesive soils, or on cohesionless soils; they are discussed separately.

2.3.1 Cohesive Soils

The basic equation for the ultimate resistance of a pile in cohesive soils may be considered as

$$Q_u = N_c \, c_b \, A_b + CL \, c_a \tag{2.3}$$

where

N_c - bearing capacity factor,

c_b - undrained cohesion at the level of the base,

A_b - area of the pile base,

C - circumference of the pile shaft,

L - length of the shaft,

c_a - average adhesion between pile and soil.

Meyerhof (1951) has shown theoretically that the value of N_c is approximately equal to 9 provided the pile is driven to a depth of atleast 5 times the pile diameter. Tomlinson (1957) observed that the adhesion expressed as percentage of the undisturbed cohesion of the clay falls with increasing stiffness of the clay - from 100 in very soft clays to 20 in very stiff clays.

Around 1960 several authors (Zeevart, a.o.) proposed the use of effective stress approach to determine the side friction in clays, which can be expressed as

$$f_s = K \, . \, \tan \delta \, \sigma_v' \tag{2.4}$$

where

f_s - unit skin friction

K - lateral earth pressure coefficient

δ - friction angle between shaft and soil

σ_v' - effective vertical stress at any depth.

An investigation by Whitaker and Cooke (1965,66) on bearing capacity of large bored piles in London clay showed that the maximum shaft resistance was reached at a displacement less than one percent of the shaft diameter, while the maximum base resistance was reached at a displacement of the order of 10 percent of the diameter of the shaft.

Williams and Coleman(1965) suggested a relation between the ultimate resistance and the lower limit of the undrained shear strength, C_{uD} at any depth D as :

$$Q_u = C \int_o^L \alpha\ C_{uD}\ dL + \pi\ R_b\ 9.\ C_{ub} \qquad (2.5)$$

where

R_b - radius of the pile base

C_{ub} - undrained shear strength at the pile base level

α - a factor with a value in the range 1.0 to 1.15

Based on total stress analysis Whitaker and Cooke (1966) furnished an expression for ultimate resistance for bored piles, with or without enlarged base, in London clay as :

$$Q_u + W_p = \pi\ d_s\ L\ \alpha\ \bar{C} + \pi\ R_b^2\ (N_c\ \gamma_s\ C_b + \gamma_s D) \qquad (2.6)$$

where

d_s - diameter of the shaft,

$\overline{C}$ & C_b - mean shear strength obtained by laboratory tests.

γ_s - the unit weight of the soil ,

D - depth of the base from ground level.

Coyle and Reese (1966) suggested an analytical procedure to compute load-settlement curves for axially loaded piles. It consists of analysing the pile as a number of elements which are considered to be in a state of equilibrium and solving for the side friction and base resistance.

Mattes and Poulos (1971) analysed elastic responses and presented curves to predict the pile movement, or the movement of the pile groups. They conducted a series of model pile tests (individual and group) in Kaoline with piles of different stiffness.

Burland (1973) suggested the use of the relation,

$$f_s = \beta \, \sigma_v' \qquad (2.7)$$

where the values of β range from 0.25 to 0.4 which were obtained from various pile load test data on clay and an average value of 0.32 for β was obtained.

Vijayvergiya and Focht (1972) (λ method) included the undrained strength, S_u in the expression to form the relation,

$$f_s = \lambda\,(\sigma'_v + 2S_u) \qquad (2.8)$$

where λ is a function of the length of the pile.

Bjerrum (1973), based on Hvorslevs' parameters φ_e, and χ proposed the following equation for side friction.

$$f_s = \mu_t(\sigma'_h \tan\varphi_e\, D_M + \chi\, P'_e) \qquad (2.9)$$

where

μ_t - coefficient for time rate effect,

σ'_h - effective horizontal stress,

D_M - mobilization factor for the frictional resistance,

P'_e - equivalent consolidation pressure.

Taking into consideration, the effect of the mobilized friction, along with the effective stress parameters, Janbu (1976) arrived at the relation

$$f_s = S_v(\sigma'_v + a) \qquad (2.10)$$

where

S_v - a function of friction angle and depth.

a - adhesion which is equal to $\bar{c}/\tan\delta$

A reduction in mobilized side friction with increased pile length has been noticed by several authors (Vijayvergiya and Focht, 1972; Janbu, 1976; Meyerhof, 1976).

Flaate and Selnes (1978) proposed an equation

$$\mu_L = (L+20)/(2L+20) \tag{2.11}$$

where μ_L is the reduction factor for the effects due to the pile length, L in feet. They suggested the equation

$$f_s = \mu_L \ \mu_t[K \ \sigma_v' \ \tan \varphi_e + \lambda \ (\sigma_v' + 5 \ S_u)] \tag{2.12}$$

where K is the coefficient of earth pressure, and φ_e and are Hvorslevs' parameters.

Vijayvergiya (1977) predicted the skin friction - pile displacement curve, based on an analytic relationship developed earlier by himself as :

$$f_s = f_{max} \ (A_o \ \sqrt{Z/Z_c} - B_o \ Z/Z_c) \tag{2.13}$$

where

f_{max} - maximum unit friction,

Z_c - critical displacement at which f_{max} is mobilized and $Z \leq Z_c$

For clays, values of A_o and B_o are 2 and 1 respectively. For drilled shafts f_{max} may be obtained from the relation

$$f_{max} = \alpha \bar{c} \tag{2.14}$$

where α is a reduction factor.

The base resistance q at any pile displacement Z could be related to maximum base resistance q_{max} as (Vijayvergiya, 1971) :

$$q = (Z/Z_c)^{1/3} \, q_{max} \qquad (2.15)$$

where

$q_{max} = N_c \, \bar{c}$ for clays

Most of the work by Poulos and Davis (1980), presented in the form of design curves, can be readily used. They presented analysis of pile groups as well as of individual piles in all types of soils.

Kraft, Focht and Amera Singhe (1981) examined the influence of pile length on frictional capacity in clays, to place the length effect in proper perspective. They considered mainly the offshore structures that are supported by open-ended pipe piles of large diameter driven to great depths. Out of various factors that contribute to length - effect, the authors felt that the major contribution is by the strain-softening phenomena and the lateral pile movements during installation.

After analysing various methods available Kraft and others concluded that the empirical relation given by Vijayvergiya and Focht (1972),(equation (2.8), Lambda method), provides the most consistent and reliable single method for estimating the axial capacity of piles in both NC and OC clays.

2.3.2 Cohesionless Soils

The classical formulae for calculating ultimate bearing capacity of piles in cohesionless soil is of the same form as equation (2.1),

$$Q_u = N_q \sigma'_v A_b + \frac{1}{2} K_s \sigma'_v \tan \delta A_s \qquad (2.16)$$

where

N_q = bearing capacity factor,

A_b - area of the base,

A_s - area of the shaft,

δ - effective friction angle between pile and soil.

Values of N_q have been established by Terzaghi and Peck (1967), Meyerhof (1951) and Berezantsev et al. (1961) based on different assumptions for different modes of failure.

Van Der Veen (1957) proposed the use of Dutch Cone-Penetrometer for the prediction of the ultimate base resistance allowing a factor of safety of 2.5.

Gregersen, Aas and Dibiagio (1973) found the pile base resistance was only half the cone-resistance for loose and medium through coarse sands in Norway. Gruteman (1973) reported that a factor of 0.75 may be used to obtain the ultimate base resistance of piles in silty sands in Russia.

Tomlinson (1977) suggested that a factor of safety of 2.5 be used to obtain the allowable base resistance from a Dutch-Cone-penetration data.

Nordlund (1963) studied the behaviour of driven piles in cohesionless soil to identify the factors affecting the frictional capacity. From further studies (1965) he concluded that the values of N_q furnished by Berezantsev best conform to the practical criteria for pile failures (Tomlinson 1977).

The skin friction in case of driven piles in cohesionless soils can be calculated by using the coefficients given by Nordlund (1965);

$$Q_s = \frac{1}{2} K_s \sigma_v' \tan \delta A_s \qquad (2.17)$$

He suggested values for δ in terms of the total friction angle φ of the soil and the K_s values with the relative density.

Tomlinson (1977) found that Nordlund's method is not valid beyond a penetration depth of 20 diameters for a straight pile.

Gregersen et al. (1973) concluded, after analysing pile load test data in loose sand, that there was no significant difference in the bearing capacity for a cylindrical and a conical pile of equal length and top diameter. He found that the bearing capacity of a pile in homogenous and loose sand deposit to increase linearly with embedded pile length.

Touma and Reese (1974) suggested that the skin friction can be calculated from equation (2.17), using a value of 0.7 for K_s and taking the effective angle of resistance of the soil as δ. They explained the total skin friction as the summation of the skin friction over the pile length. The ultimate resistance can be calculated as that resistance which results in a settlement of 25 mm, given by the expression

$$Q_B = (\frac{50A}{B})q_u \tag{2.18}$$

In metric units the values for q_u varies as zero in loose sand and 3830 kg/cm^2 in dense sand.

Potyondy (1961) on the basis of general friction theory suggested that in the case of skin friction, stress-strain curves can be derived from

$$\frac{f_s}{\sigma_v} = \tan\delta\,[1 - \exp(-k\,\frac{S}{S_o - S})] \tag{2.19}$$

where

k - a constant for the soil

S - pile movement

S_o - the critical pile movement at which the full shaft friction is being mobilized.

Various experiments were conducted to find the value of δ in terms of φ of the sand, and the value of S_o, expressed as percent of pile diameter.

Coyle and Reese (1966) found the best relations for shaft resistance and base resistance as :

$$f_s = \sigma'_v K \tan \delta \tag{2.20}$$

$$q_b = \sigma'_v N_q \tag{2.21}$$

for reasonably accurate evaluations of K, δ and N_q.

Chin (1972) suggested the inverse slope for predicting the ultimate bearing capacity of piles for pile tests carried out to sufficiently large movements.

Work on separation of the total ultimate load into skin resistance and base resistance is reviewed in Chapter 4.

2.4 LATERALLY LOADED PILES AND PILE GROUPS

Bulk of the work done on laterally-loaded piles were based on lateral earth pressure theory. But the research has not yielded any simple design method which can be universally applied to any soil, or type of pile. Research was done on the basis of bending moment theories (Poulos and Davis, 1980). Model tests were conducted by several authors (Kubo, 1965; Poulos and Davis, 1980; and others) to find expressions for the ultimate bearing capacity of piles under lateral loads.

The behaviour of a group of piles considerably differ from that of individual piles. Much research has been done

on model pile groups and fullscale pile groups. It is found that the total capacity of the group may not be the sum of the individual capacities. Research was conducted to evaluate efficiency of pile group with respect to individual piles.

CHAPTER 3

MODEL STUDIES

3.1 INTRODUCTION

The prediction of pile capacities is a continually recurring problem for engineers. The most conventional procedure for finding out the capacity of a pile is to conduct a field loading test at the site and correlate the information thus obtained to predict the capacity of other piles at the same site. From a field test, only the overall response is available and it is difficult to separate shaft and base resistances.

Model tests conducted in the laboratory may be helpful to arrive at the nature of the load-settlement relation, and can be directly used to predict the field capacities. Model tests on pile for shaft resistance or base resistance alone can be conducted separately. Coyle and Reese (1966) suggested to use a triaxial test apparatus to conduct model tests on piles for shaft resistance only.

A series of model tests were conducted in remoulded clay and medium dense sand to simulate a segment of a long pile. The sands tested were calcareous derived from Gujarat. The clays tested were also received from Gujarat.

The present study was done to identify the factors that affect the shaft resistance of a pile.

3.2 BRIEF REVIEW OF EARLIER WORK

Potyondy (1961) carried out several laboratory shear box tests with various construction materials, surface conditions, various types of soil at different moisture contents and under different normal loads between the friction surfaces. He used strain controlled and stress controlled shear box to study the influence of various factors that affect the shearing resistance and concluded that the shearing resistance varied with the change of moisture content. For rough concrete piles he found that the value of interface friction angle reached almost equal to the angle of shearing resistance of the soil and in every case skin friction was lower than the shear strength of the soil.

Whitaker and Cooke (1961) conducted a series of tests using small piles in London clay and found that the shaft resistance was mobilized at small movements. Eventhough the tip of the pile was in the soil mass, the resistance offered by the tip is appeared to have been neglected.

Coyle and Reese (1966) used conventional triaxial cell with proper modifications for conducting large scale model pile tests in clay, in such a manner that the effect of depth could be simulated. Tests revealed that a definite linear relationship exists between load transfer and pile movement for small movements. The shaft resistance reached a peak value at smaller movements (in the range of 9 to 13 percent

of the pile diameter) and then dropped off to a constant residual value at larger movements.

Coyle and Sulaiman (1967) performed the same type of model tests on saturated sands using the same experimental set up used by Coyle and Reese. In the case of saturated sand the skin friction reached a maximum value at smaller movements of the order of 3 to 5 percent of the pile diameter and then dropped off. It was observed that the skin friction continued to increase, but at a small rate, for larger pile movements.

The results of a series of model pile tests in Kaoline conducted by Matts and Poulos (1971) showed a good agreement between predicted and observed pile settlements where the predictions were based on elastic theory. To study the effect of pile compressibility on the pile movement, two piles made from a plastic material were also tested. They concluded that the elastic theory, if used judiciously, can adequately predict the settlement of pile foundation.

Hanna and Tan (1973) conducted compression loading tests on long model piles of diameter 24.4 mm and length 1800 mm in compacted sand. The shaft friction was mobilised when the pile movement was 2.5 percent of the pile diameter. As the pile tip was resting in the soil, the tip could have offered resistance which appears to have been neglected.

Clemence and Brumund (1975) conducted a large scale model test of drilled concrete pier in sand. The interface friction angle was determined from a series of direct shear tests with 2.5 inch diameter circular direct shear test device. The model pier was instrumented with strain gauges at different levels to study the effect of the effective lateral pressure. He observed that the shaft resistance-displacement relation is hyperbolic but the asymptotic value gave a slightly larger value.

Bee (1975) conducted a series of laboratory rod-shear tests on silty clay with 1/2 inch diameter steel rods and came up with a conclusion that the skin friction is primarily a function of the effective lateral pressure between the pile and the surrounding soil, and secondarily related to the roughness of the pile, the soil type and very much insensitive to the rate of loading. The movements at peak adhesion were typically between 1 to 4 percent of the pile diameter. A unique relationship was found to exist between moisture content and the logarithm of effective lateral pressure, as well as shear strength and interface strength. It was found that the interface strength was approximately equal to the unconfined compressive strength of the surrounded soil for smooth and rough piles.

Holmquist, Thompson and Matlock (1976) conducted a series of model pile tests on NC and OC clays to study the factors that affect the resistance-displacement relationship, which include installation effects, confining pressure, pile surface conditions and local consolidation due to driving. The peak static skin friction was found to be equal to 1.5 times the remoulded undrained vane shear strength. They describe the lab test, referred to as triaxial rod shear tests, as an alternative method which is less expensive compared to field tests to find out load-settlement relationships. They concluded that for static loading the amount of time allowed for local consolidation affects the pile capacity considerably.

3.3 EXPERIMENTAL SET UP

The experimental set up used for the work is very similar to the one used by Coyle and Reese (1966,67). An element of a long pile embedded at depth was considered and simulated by a pipe of small diameter surrounded by soil. A large triaxial cell was used which can accommodate specimens of 10 cm diameter and 20 cm height. The whole system consists of the following :

3.3.1 Triaxial Cell

A cell of 165 mm diameter and 300 mm in height was used with frictionless piston assembly. It had an outlet to allow drainage and an inlet to connect to cell pressure.

Figure 3.1 shows the details of the cell with the soil specimen.

3.3.2 Self Compensating Mercury Control System

This system was used to apply and maintain the cell pressure during the test. The pressure of water in the triaxial cell results from the difference in the level between the mercury surfaces in two small cylinders connected by a thin flexible pressure tube as described by Bishop and Henkel (1962).

3.3.3 Strain Controlled Loading System

A standard loading machine in which desired strain rates could be set was used to conduct the experiments. It consists of a fixed loading frame with a provision for proving ring to measure axial load and a base on which triaxial cell can be placed and which can be moved up and down at any rate desired using a gear and toothed wheel configuration.

3.4 SOILS CONSIDERED AND CONVENTIONAL TEST DETAILS

Two types of sands and clays were considered for the study. Tests were conducted to classify and to find out the strength parameters of these soils.

3.4.1 Soils Considered

Types of sand considered are classified as sands A and B and clays tested are denoted as clays C and D. Geographical location, General appearance and the depth from which the samples are collected are given in Table 3.1.

Table 3.1
Description of the Soils tested

	Soil Type	Location	Appearance	Depth from ground level metres
Sand A	Calcareous	Near Okha Light House, Gujarat	rounded and angular particles	0
Sand B	Calcareous	South east Okha Gujarat	mostly angular particles	0
Clay C		Dwaraka Bet, Gujarat	yellow color soft	3
Clay D	Lateritic clay	Dwaraka Bet Cliff Gujarat	red color	3.25

3.4.2 Test Details

3.4.2.1 Grain size

Seive analyses for sands A and B were done and grain size distribution is shown in Fig. 3.2 . Sand B is uniformly graded while sand A is slightly well graded. Hydrometer

analysis for clays C and D were conducted and test results are shown in Fig. 3.3.

3.4.2.2 Liquid limit and plasticity index

Liquid limit and plastic limit of clays were determined by conventional procedures. Clay C is having a liquid limit of 48 percent and PI of 23 percent while clay D is having liquid limit of 50 percent and PI of 23 percent.

3.4.2.3 Triaxial tests

Consolidated drained tests were conducted for sand A and sand B using specimens with a density of 1.65 gms/cc. Consolidated undrained tests with pore pressure measurements were performed for clays. Test data for sands is shown in Fig. 3.4 and results in Fig. 3. 5. Sand A is having a friction angle of 44 degrees while that for sand B is 45 degrees. Figs. 3.6 and 3.7 give the test data and results for clays C and D. The effective friction angles for clays C and D are 23 degrees and 32 degrees respectively.

3.4.2.4 Direct shear tests

Direct shear tests were conducted for sand A and sand B in loose condition (unit weight = 1.49 gm/cc). Results are shown in Fig. 3.8. The friction angles for sands A and B in loose state are 36 degrees and 37 degrees respectively,

which are very less compared to that for dense sand (unit weight 1.65 gm/cc) obtained from triaxial tests.

3.4.2.5 Consolidation tests

The results of Oedometer tests on sands A and B are given in Fig. 3.9. The compressibility index C_c for sands A and B are 0.028 and 0.092 respectively. Oedo meter tests were conducted for clays C and D and the data are given in Fig. 3.10. e-log p curves for Clays C and D are shown in Fig. 3.11. Clay C is having a compressibility index C_c of 0.256 while that for Clay D is 0.239. The coefficient of consolidation, C_v for clay C is 0.746×10^{-2} cm^2/sec for a pressure increment of 2 km/cm^2 to 4 kg/cm^2. C_v for clay D is found to be 0.8563×10^{-2} for a pressure increment of 1.5 kg/cm^2 to 3.5 kg/cm^2.

A summary of the test results on sands and clays are given in Table 3.2.

3.5 MODEL TESTS

3.5.1 Description of the Cell and Other Provisions

Tests were carried out as described by Coyle and Reese (1966) with slight modification in the test set up. Fig. 3.1 shows the details of the triaxial cell used. When the confining pressure is applied it is hydrostatic and acts in all directions. The pile was held at the centre of the specimen, with the aid of two aluminium rings, one at the

Table 3.2 Conventional test results for sands A and B

Soil	G_s	Uniformity coefficient	LL	PI	C_v cm^2/sec	C_c	φ' degrees
Sand A	2.677	2.477	-	-	-	0.028	44,36*
Sand B	2.71	1.738	-	-	-	0.0256	45,37*
Clay C	2.593	-	48	23	0.746×10^{-3}	0.256	23
Clay D	2.646	-	50	23	0.856×10^{-3}	0.239	32

*Values from direct shear tests

top and the other at the bottom. The tip of the pile was not constrained from movement and sufficient space was provided for the movement of the pile with the aid of a pedastal as shown in Fig. 3.1. The pile was free to move through the top and bottom aluminium rings with least friction and at the same time no soil particles were allowed to escape through the gap between the pile and the rings. Entering of water into the specimen from bottom was prevented by the acrylic base used. To prevent the leakage of water from the top, a brass cap with a rubber membrane fixed to it and stretched over the top aluminium plate was used (Fig. 3.1). 'O' rings were used to prevent leakage through the gap between the rubber membrane and the top and bottom plates. Provisions were made for drainage of water through the base plate (Fig. 3.1) during consolidation and pile loading. During testing, the confining pressure was acting on the specimen from all directions and only the pile was being axially loaded.

3.5.2 Types of Piles Used

Aluminium pipes of outer diameter 21.76 mm and length of 240 mm, and Brass pipes of outer diameter 22.2 mm and length of 240 mm were used. The length of that portion of the pile in contact with soil was 189 mm. The surface of an aluminium pipe of outer diameter 21.76 mm was made rough by sticking uniform sand of grain size 0.8 mm on the surface

with araldite to make a rough pile of mean outer diameter of 23.4 mm. Plate No. 3.1 shows the piles used for testing.

3.5.3 Preparation of Specimen

A. Sand Specimens

Sand specimens of dry density 1.63 gm/cm^3 were prepared in dry condition using a split former in two halves with arrangement to create small suction. The pile was held in position and the weighed sand was poured in layers and compacted to the required density. Number of layers and amount of compaction to attain the required density were found out by trial and error. The size of the specimens used were of 100 mm diameter and 189 mm in length. Setting up of specimen is shown in Fig. 3.1.

B. Clay Specimens

Remoulded clay specimens were obtained by resedimenting the clay slurry. Two perforated brass cylinders of length 300 mm and 100 mm inside diameter were put together to form one mould. Provisions were made to hold the pile in the centre of the cylinder. The slurry was then poured, and allowed to sediment and consolidate under desired vertical pressure. The samples were taken out after seven days by which time consolidation was seen to be nearly complete. They were cut into required size and were used for testing. The size of the specimens used were 100 mm in diameter and of 189 mm long.

3.6 TEST PROGRAMME

Tests were conducted at different confining pressures. Loading-unloading and then reloading tests were also performed under different confining pressures. In the case of clay, specimens were tested at different rates of loading to see the effects of rate on shaft resistance under a given all round pressure. Rough piles in sand A were tested under two different confining pressures.

3.7 RESULTS AND DISCUSSIONS

In this section the load-settlement behaviour of the test piles have been presented. The results are categorised with respect to the type of the soil.

3.7.1 Sand A

Tests were conducted under confining pressures of 1.0, 1.5, 2.0 and 3 kg/cm^2 at an average strain rate of 0.036 mm/min. The pile material used was aluminium. The load-displacement curves are presented in Fig. 3.12. The curves show that the shaft resistance is having a linear relationship with the pile movement and it reaches a peak value after which it dropped off to a residual value. Fig. 3.13 gives plots between non-dimensional shaft resistance (ratio of shaft resistance to confined pressure) and movement as a percentage of pile diameter. The shaft resistance reached a peak value at a pile movement of 1.22 to 1.45 percent

of the pile diameter, except for the test under 3 kg/cm^2 confining pressure for which the pile movement was 4.2 percent.

It can be seen that the peak pile-soil friction angle δ is nearly same for all confining pressures, but slightly increases with the increase of confining pressure. The value of the peak pile-soil friction angle ranges from 10 to 13 degrees where as the residual friction angle is 9.5 degrees. The ratio of peak pile-soil friction angle with the internal friction angle of the soil, φ, range from 0.23 to 0.29.

Two cyclic load tests (loading-unloading-reloading) were performed under confining pressures of 1 kg/cm^2 and 1.5 kg/cm^2. The results are presented in Figs. 3.14 and 3.15. An increase in the initial modulus is noted for reloading. A considerable amount of elastic recovery (about 50 percent) is also noticed. The values of the peak pile-soil friction angle are also increased by the order of 2 degrees.

Rough piles which were made as described earlier in this chapter were tested under confining, pressures 0.5 kg/cm^2 and 1 kg/cm^2. Fig. 3.16 gives the test results which indicate a considerable increase in the shaft resistance. The failure undoubtedly occurred not along the soil-pile interface but somewhere away from the circumference of the pile. The ultimate shear strength of the soil might have been mobilised away from the pile surface.

3.7.2 Sand B

Tests on sand-B were conducted under confining pressures 1 kg/cm^2, 1.5 kg/cm^2 & 2 kg/cm^2. The tests were conducted at an average strain rate of 0.036 mm/min. The results are presented in Figs. 3.17 and 3.18. In the case of sand B the peak value of the shaft resistance was attained at a pile movement ranging from 1.78 to 4 percent of the pile diameter. Here also an increase in shaft resistance with increase in confining pressure was observed. The peak pile-soil friction angle and the residual friction angle increased with confining pressure. The range of the peak and the residual friction angles are 15.7 to 16.7 degrees and 8.5 to 10.6 degrees respectively. The ratio of the peak pile-soil friction angle to the φ of the soil is only 0.36 (average) whereas the values for other sands (eg. silica sands) is reported to be more than 0.5 (Potyondy, 1961).

A cyclic test was performed on sand B under a confining pressure of 2 kg/cm^2 which led to an increase in initial modulus for the second loading. A peak pile-soil friction angle of 25 degrees, which is remarkably larger than that of the first loading under the same confining pressure, was noticed. The results are presented in Fig. 3.19.

The graph of peak shaft resistance Vs confining pressure for both sands A and B is summarised in Fig. 3.20. The relation between the residual shaft resistance and confining pressure is also given in the same figure.

3.7.3 Clay C

For clay C tests were conducted to see the effects of confining pressure on shaft resistance. The tests were conducted with the specimen kept for consolidation under the respective confining pressures for a period of two days for cell pressure variation of 0.5 kg/cm^2, 0.75 kg/cm^2 and 1 kg/cm^2. Fig. 3.21 gives shaft resistance as a function of pile movement under various confining pressures. Ratio of shaft resistance to confining pressure as a function of pile movement expressed as percentage of the pile diameter is presented in Fig. 3.22. The peak resistances were reached at an average pile movement of 4.8 percent of pile diameter. It can be seen from Fig. 3.22 that shear strength mobilization along the soil pile interface decreases with increase in confining pressure. For example for confining pressures 0.5 kg/cm^2 and 1 kg/cm^2 the ratios are 0.42 and 0.303 respectively.

Two loading-unloading and then reloading tests were conducted on clay C for confining pressures of 0.75 and 1 kg/cm^2. The results are presented in Fig. 3.23. The maximum shaft resistance was higher in cyclic loading tests than that in single-loading test. The tests were conducted at an average strain rate of 0.0025 mm/min.

The ratio of the peak-pile-soil friction angle to φ' varies from 0.69 for a confining pressure of 1 kg/cm^2 to 0.863 for 0.5 kg/cm^2.

3.7.4 Clay-D

Under a confining pressure of 0.75 kg/cm^2 tests were conducted at different strain rates which in turn caused a difference in the quantity of water drained out from the specimen. The tests were performed after the specimens were kept under the same chamber pressure for consolidation for a period of two days. Some tests were conducted at a higher strain rate under the same confining pressure of 0.75 kg/cm^2 without allowing for consolidation, and one of them is presented. Fig. 3.24 shows the results the results of the tests conducted at different rates under a confining pressure of 0.75 kg/cm^2. The test which was done without consolidation and at a higher strain rate gave very small value for the shaft resistance, indicating undrained failure. The higher rates of loading led to smaller values of shaft resistance because of high water content. The water content measured after the tests and the rates at which the tests were done are also indicated in Fig. 3.24.

Experiments were conducted under confining pressures of 0.5, 0.75 and 1 kg/cm^2 at an average strain rate of 0.0025 mm/min The test results are shown in Fig. 3.25 with shaft resistance

as a function of pile movement for different confining pressures. Fig. 3.26 gives the ratio of shaft resistance to confining pressure as a function of pile movement expressed as percentage of pile diameter. As in the case of the sands and clay C the shaft resistance reached a peak value and then dropped off to a constant residual value at larger pile movements. It is evident from Fig. 3.25 or 3.26 that the shaft resistance is having a linear relationship with the pile movement for small pile movements. It was also observed that increase in confining pressure led to a decrease in mobilization of shear strength along the pile-soil interface. For example for confining pressures 0.5 and 1 kg/cm^2, the ratios are 0.361 and 0.284 respectively.

Cyclic loading tests were performed on piles in clay D under confining pressures 0.75 and 1 kg/cm^2, the results of which are presented in Fig. 3.27. The ratio of peak pile-soil friction angle varies from 0.53 to 0.71 (higher value for lower confining pressure).

A summary of the test results on clays C and D is presented in Fig. 3.28.

The results of all the model tests are summarised in Tables 3.3a and 3.3b.

Table 3.3a Summary of model pile tests on sand A and B

Soil	Pile	Strain rate mm/min	σ_3 kg/cm2	S_o pile dia	δ degree	δ/φ	K kg/cm	Remarks
Sand A	Aluminum	0.036	1	1.22	10.2	0.232	94	
$\varphi = 44^o$	-do-	0.036	1.5	1.23	12.1	0.275	159	
	-do-	0.036	2	1.45	12.3	0.28	159	
	-do-	0.036	3	4.22	12.7	0.289	138	
	-do-	0.036	1	0.78	14.4	0.327	149,300*	*for the re-loading
	-do-	0.036	1.5	1.18	14.8	0.336	170,377*	-do-
	Rough	0.07	0.5	12.33	44.0	1	170	soil-soil failure.
	-do-	0.07	1	13.56	44.0	1	300	-do-
Sand B	Aluminum	0.036	1	1.79	15.8	0.350	110	
$\varphi = 45^o$	-do-	0.036	1.5	2.83	16.0	0.356	110	
	-do-	0.036	2	4.00	16.2	0.36	117	
	-do-	0.036	2	1.90	25	0.568	288,523*	*for re-loading

Table 3.3b Summary of Model Pile Tests on Clays C and D

Soil	Pile	Strain rate mm/min	σ_3 kg/cm^2	S_o pile dia	δ degree	δ/φ	K kg/cm	Remarks
Clay C		0.0025	0.5	6.32	19.86	0.863	110	
$\varphi = 23^o$		0.0025	0.75	3.63	16.45	0.7152	150	
		0.0025	1.0	4.64	15.87	0.689	136	
		0.0025	0.75	4.87	17.146	0.745	123,180*	*for re-loading
	Brass	0.0025	1.0	3.88	16.48	0.7165	150,230*	
Clay D		0.0025	0.5	4.46	22.78	0.7118	137	
$\varphi = 32^o$		0.0025	0.75	4.48	18.33	0.573	163	
		0.0025	1.0	4.96	16.827	0.526	145	
	Brass	0.003	0.75	3.54	15.8	0.4936	-	
		0.004	0.75	0.87	3.64	0.11375	-	
		0.0035	0.75	2.90	15.073	0.471	-	
		0.0025	0.75	3.63	14.91	0.466	145,189*	*for re-loading

3.8 CONCLUSIONS

1. The ratio of the peak pile-soil friction angle to the internal friction angle φ for calcareous sands A and B are found to be very small. The values range from 0.23 to 0.29 for sand A, and it is from 0.350 to 0.36 for sand B whereas the values for silica sands tested and reported are greater than 0.5.

2. Sand A gave an average maximum pile-soil friction angle of 12 degrees while sand B gave a value of 16 degrees, eventhough the internal friction angle φ for both the sands are approximately same (45 degrees). The particles of sand A are rounded while that of sand B are angular. Also sand A consists of larger percentage of fines than sand B. Both these reasons may account for lesser frictional resistance for piles in sand A compared to that of sand B.

3. The cyclic loads on sands A and B gave an average elastic recovery of 52 percent for the initial part of the loading. The peak friction angle increased by about 2^{o} for sand A and about 8^{o} for sand B due to the effect of cyclic loading.

4. The mobilization of full shaft resistance occurs at a pile movement of 1.2 to 1.5 percent of the pile-diameter for pile in sand A except in one case (for $\sigma_3 = 3\ kg/cm^2$) in which it is 4.5 percent. In the case of sand B the

mobilization occurs at a pile movement of 1.78 to 4 percent of the pile diameter.

5. For clays C and D the peak shaft resistance is mobilized at a pile movement of 3.0 to 6.5 percent of the pile diameter.

6. The ratio of the peak pile-soil friction angle to φ' for clay C varies from 0.69 for a confining pressure of 1 kg/cm^2 to 0.863 for 0.5 kg/cm^2. In the case of clay D the variation is from 0.53 for 1 kg/cm^2 to 0.71 for 0.5 kg/cm^2.

7. At lower strain rates shaft friction mobilized is larger because of the increased time for drainage of water from the specimen during the test.

8. Rough pile gives significantly large capacities in sand.

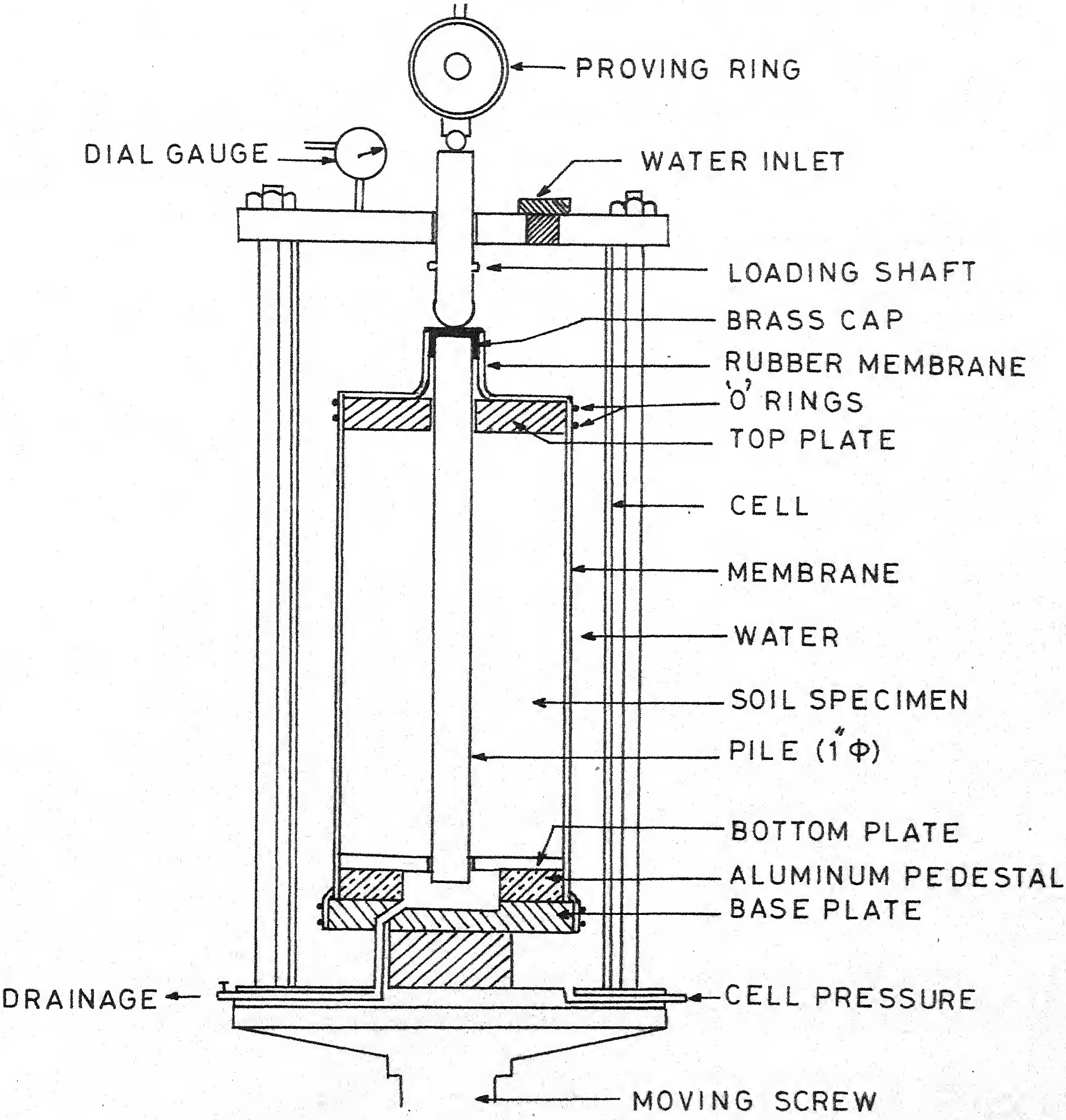


FIG. 3·1 TRIAXIAL CELL USED FOR TESTING

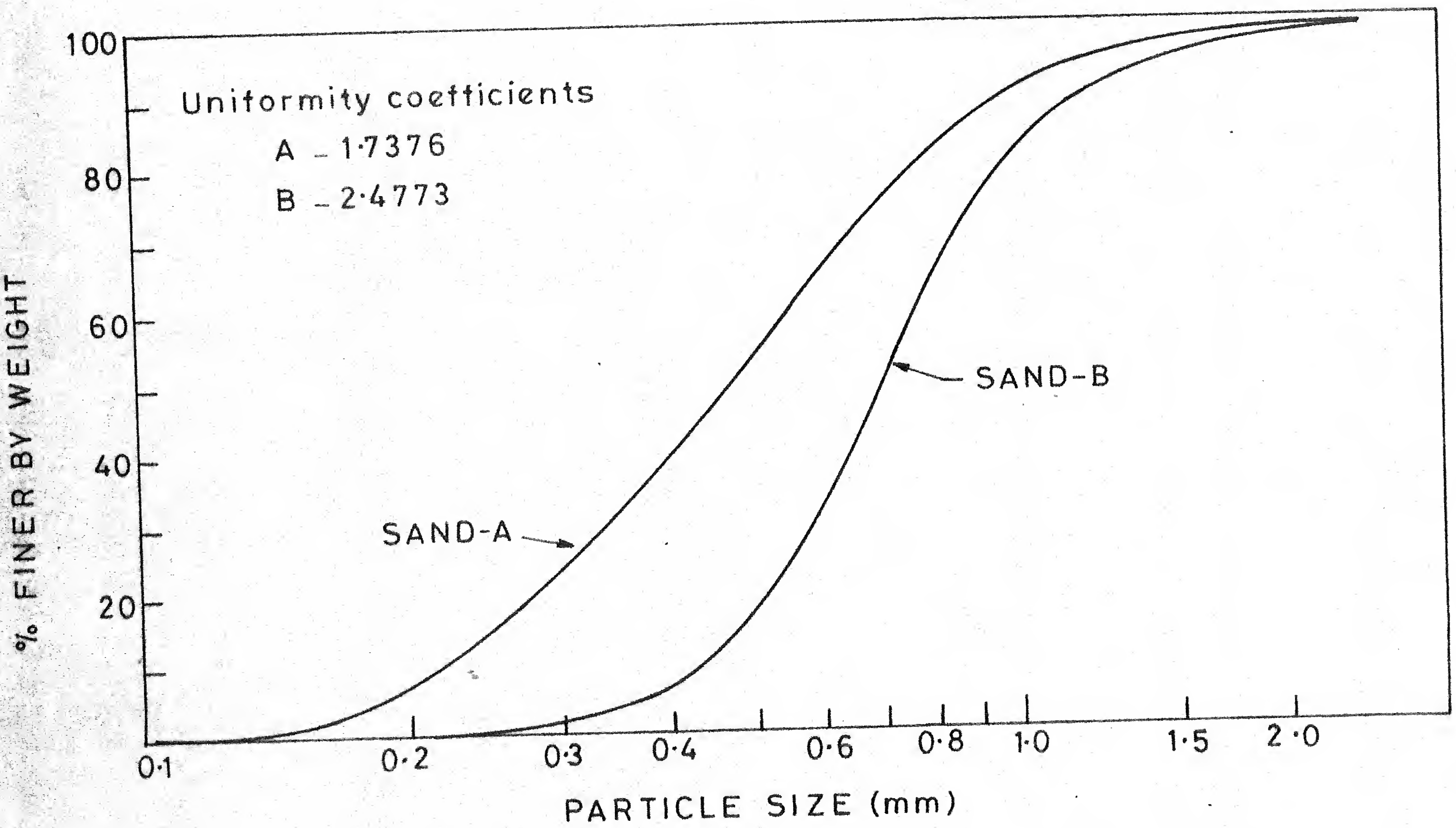


FIG. 3·2 GRAIN SIZE DISTRIBUTION OF SANDS A & B

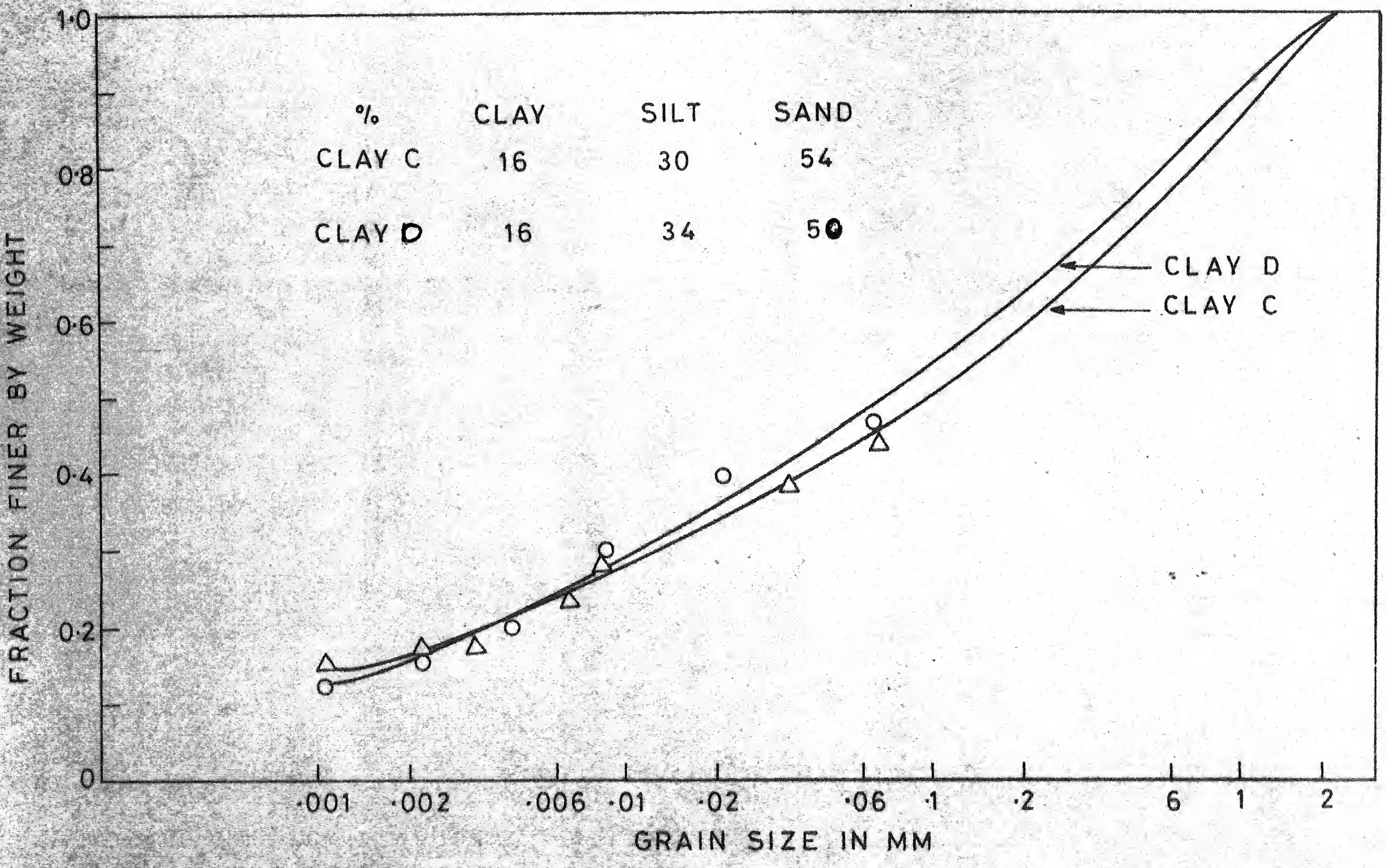


FIG.3·3 GRAIN SIZE DISTRIBUTION CURVE FOR CLAYS C & D

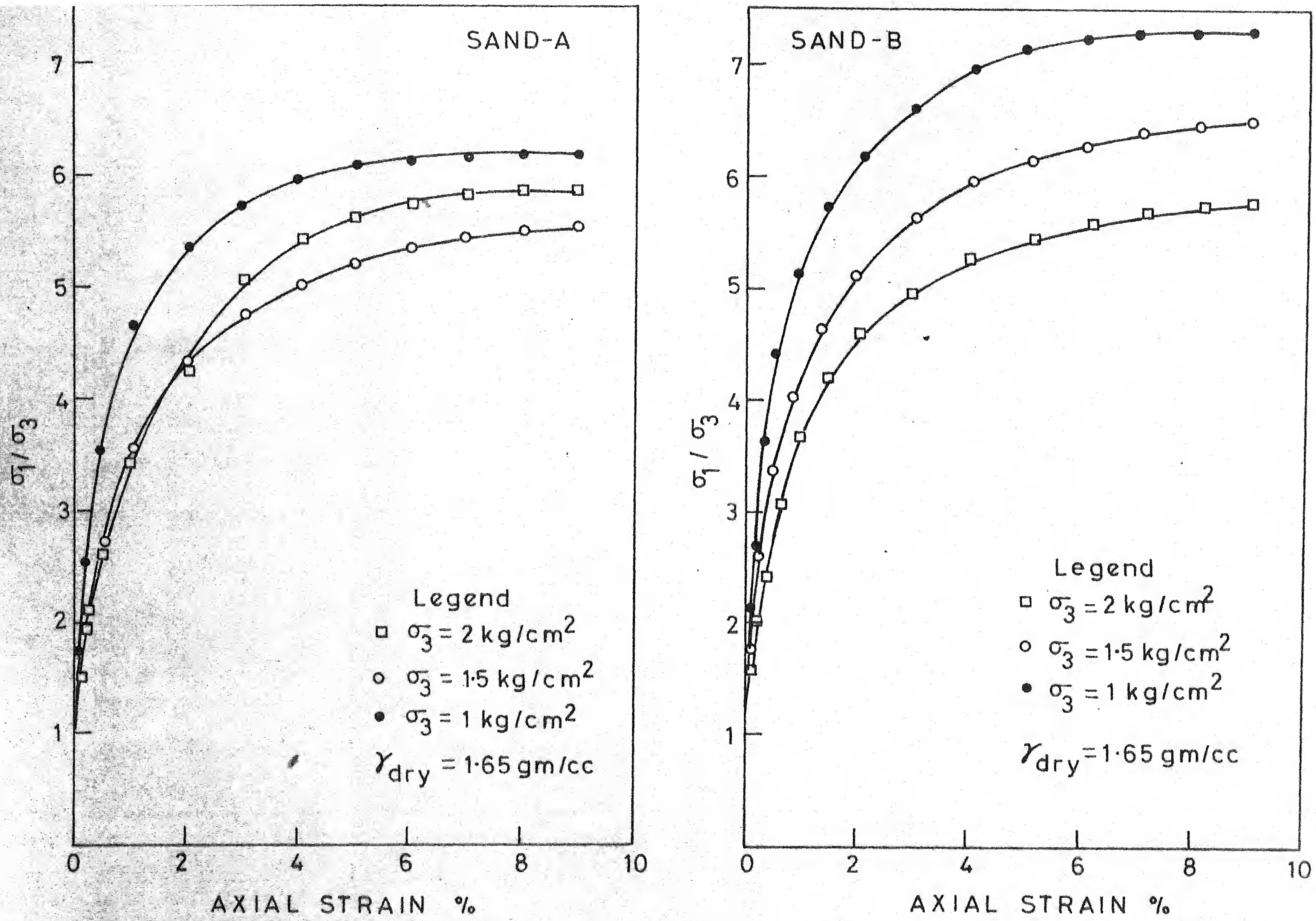


FIG. 3·4 DRAINED TRIAXIAL TEST DATA FOR SANDS A & B

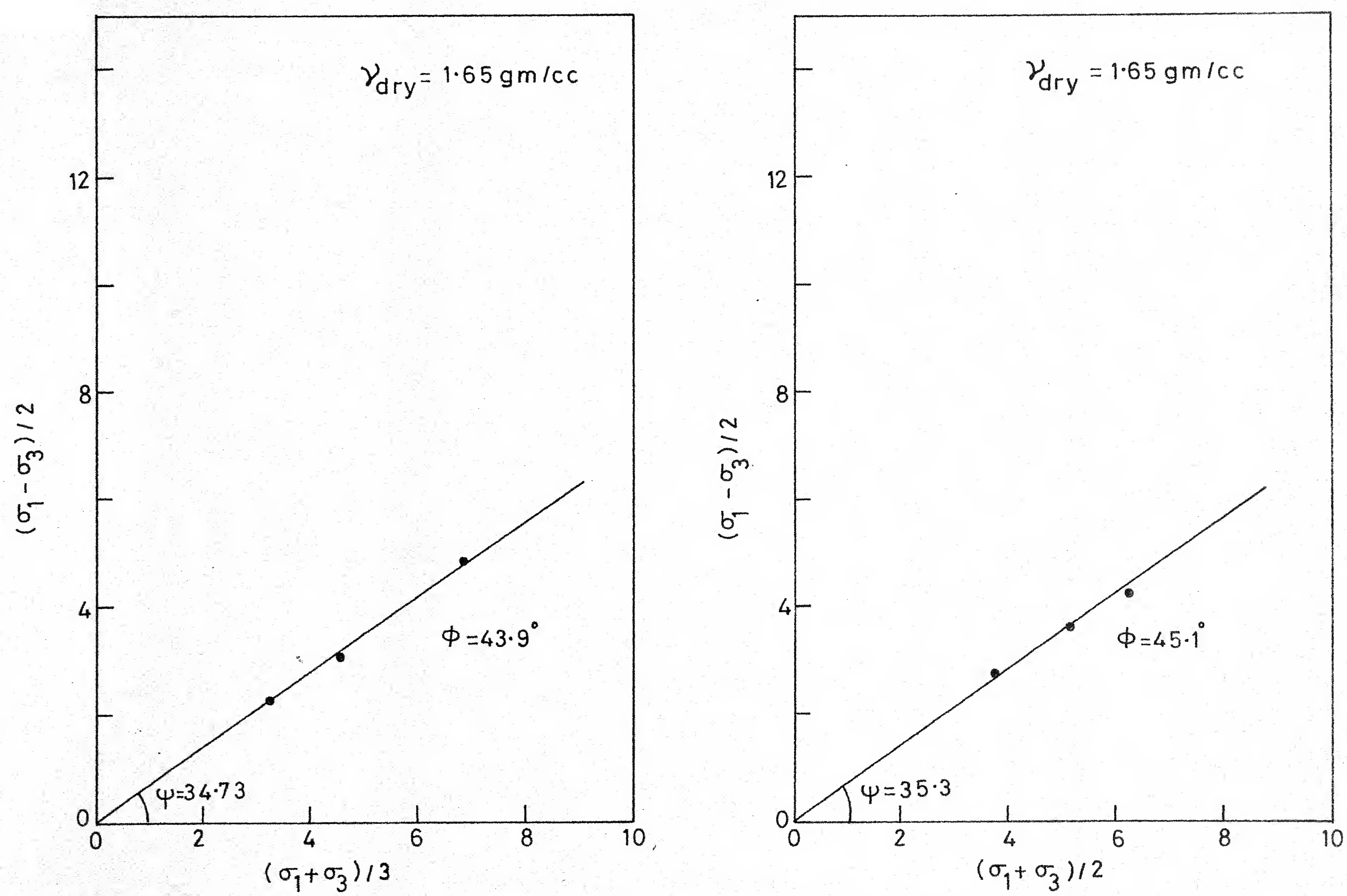


FIG. 3·5 DRAINED TRIAXIAL TEST RESULTS FOR SANDS A & B

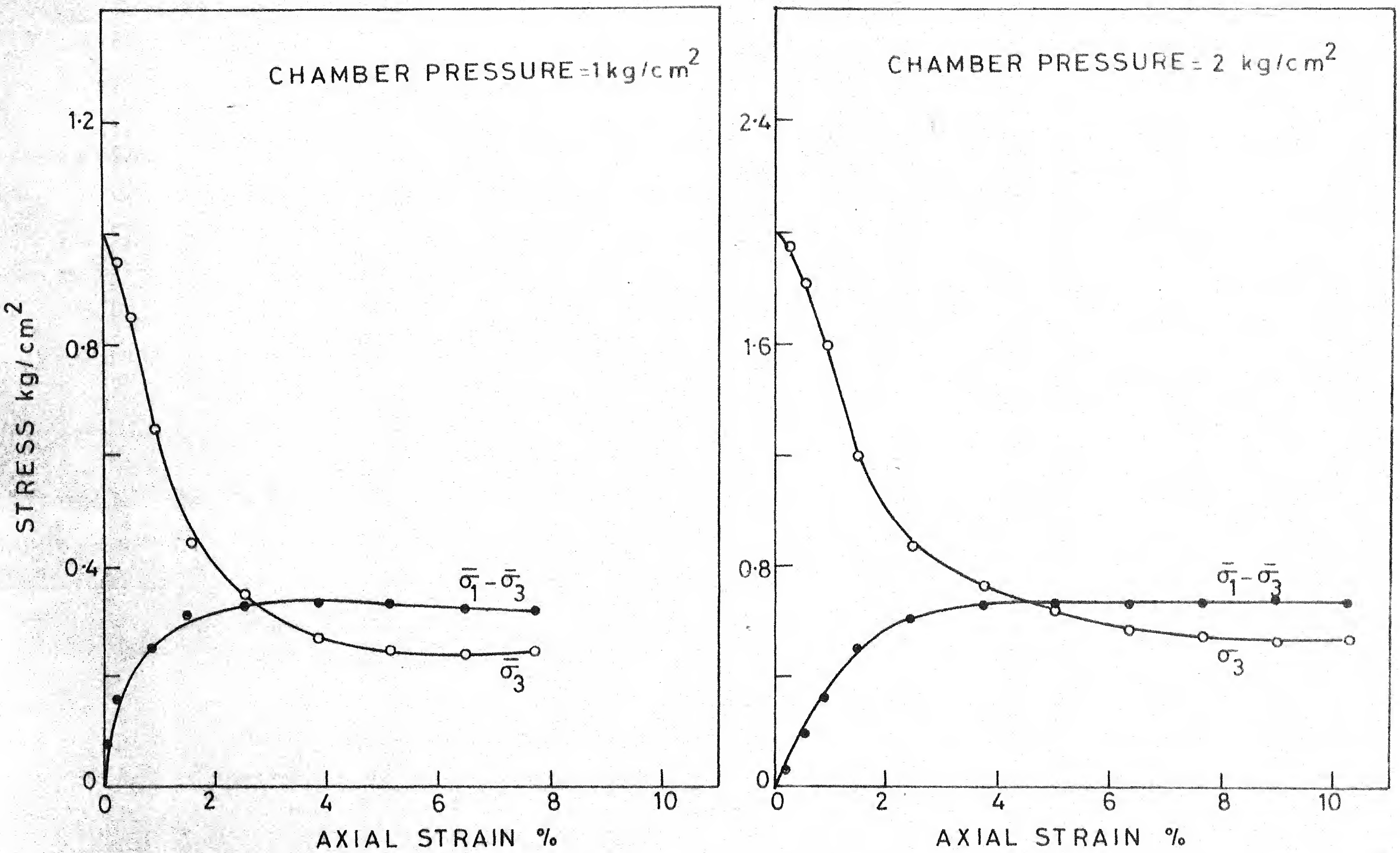


FIG. 3·6 C-U TEST DATA FOR CLAY C

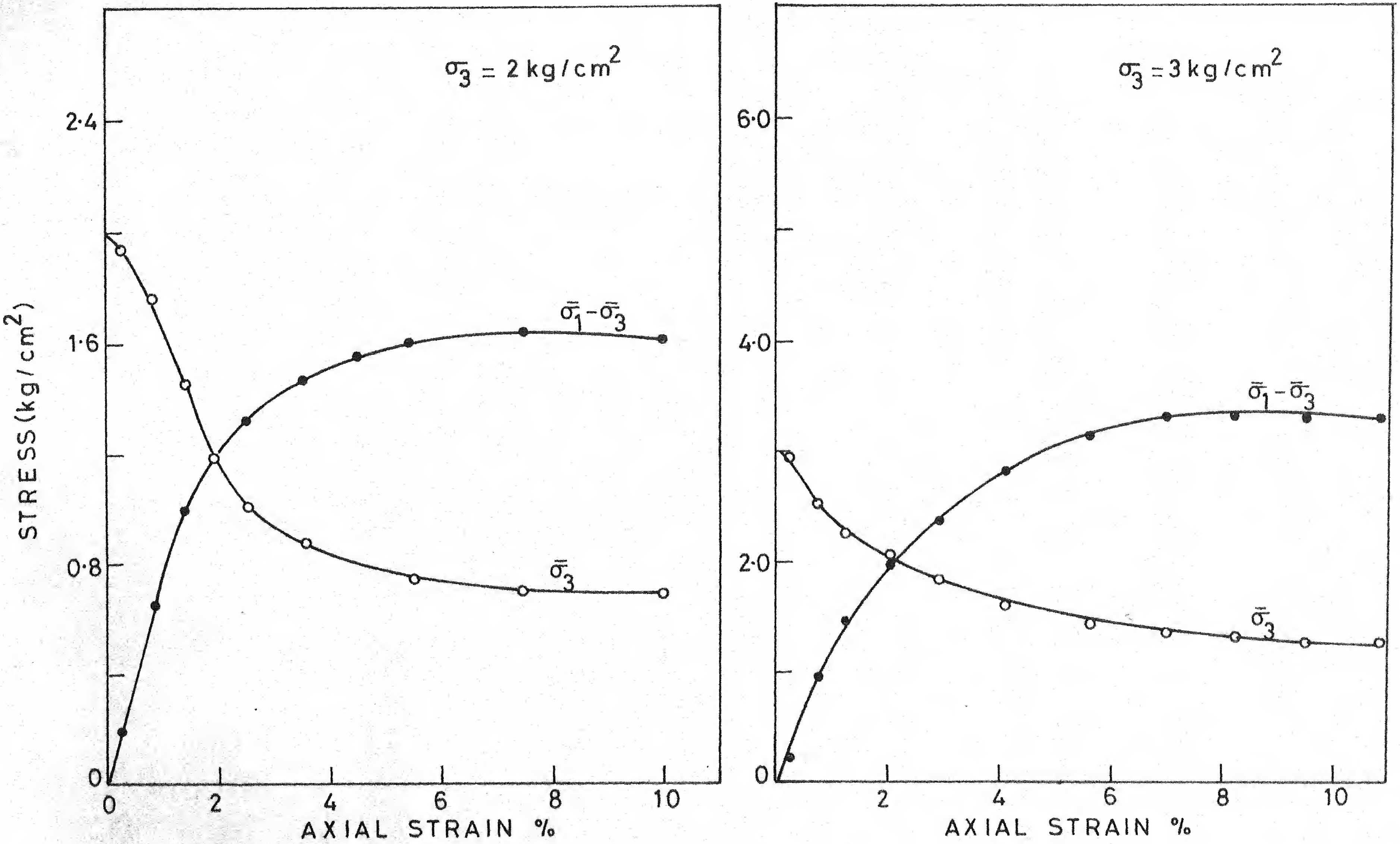


FIG.3·7 C-U TEST DATA FOR CLAY D

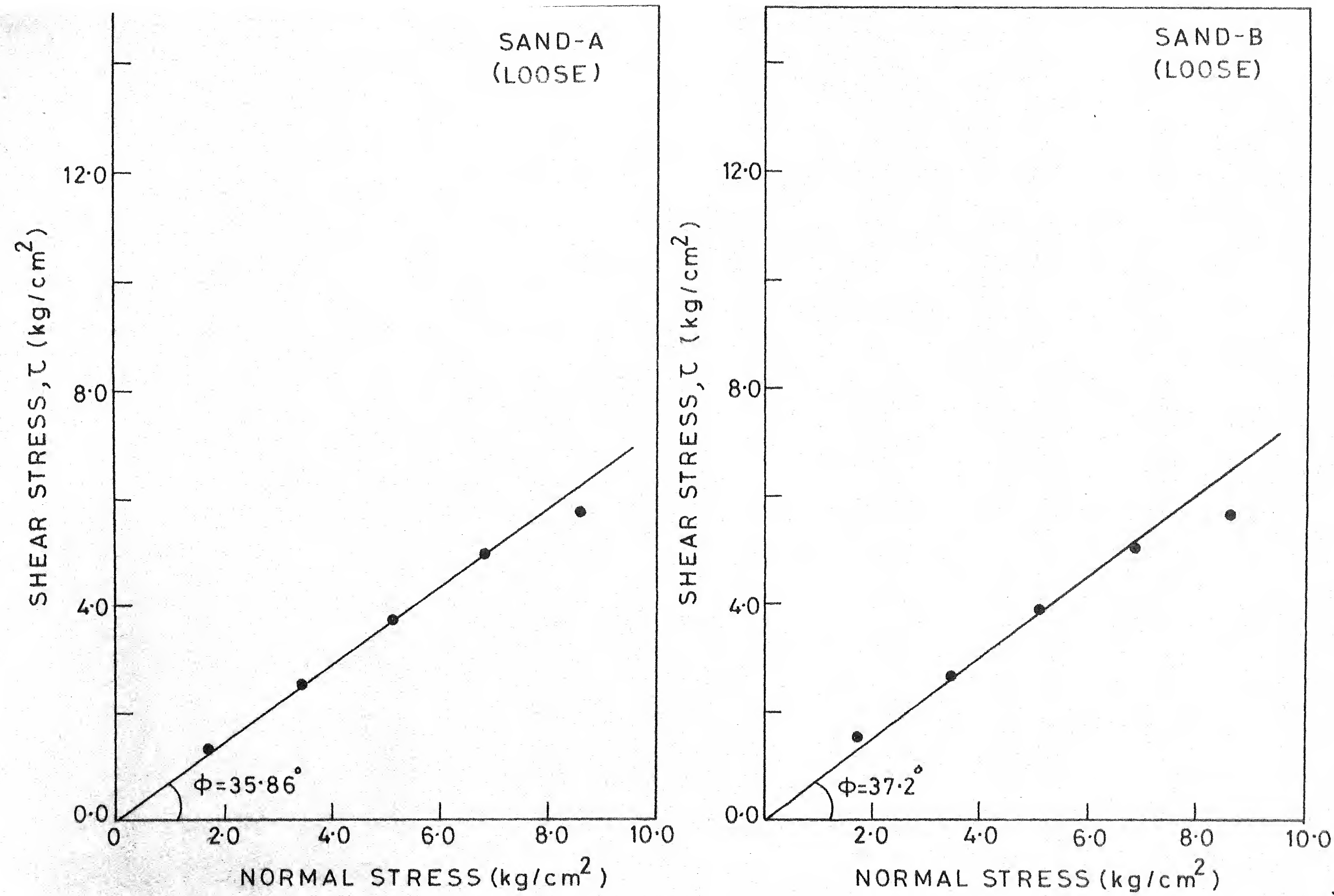
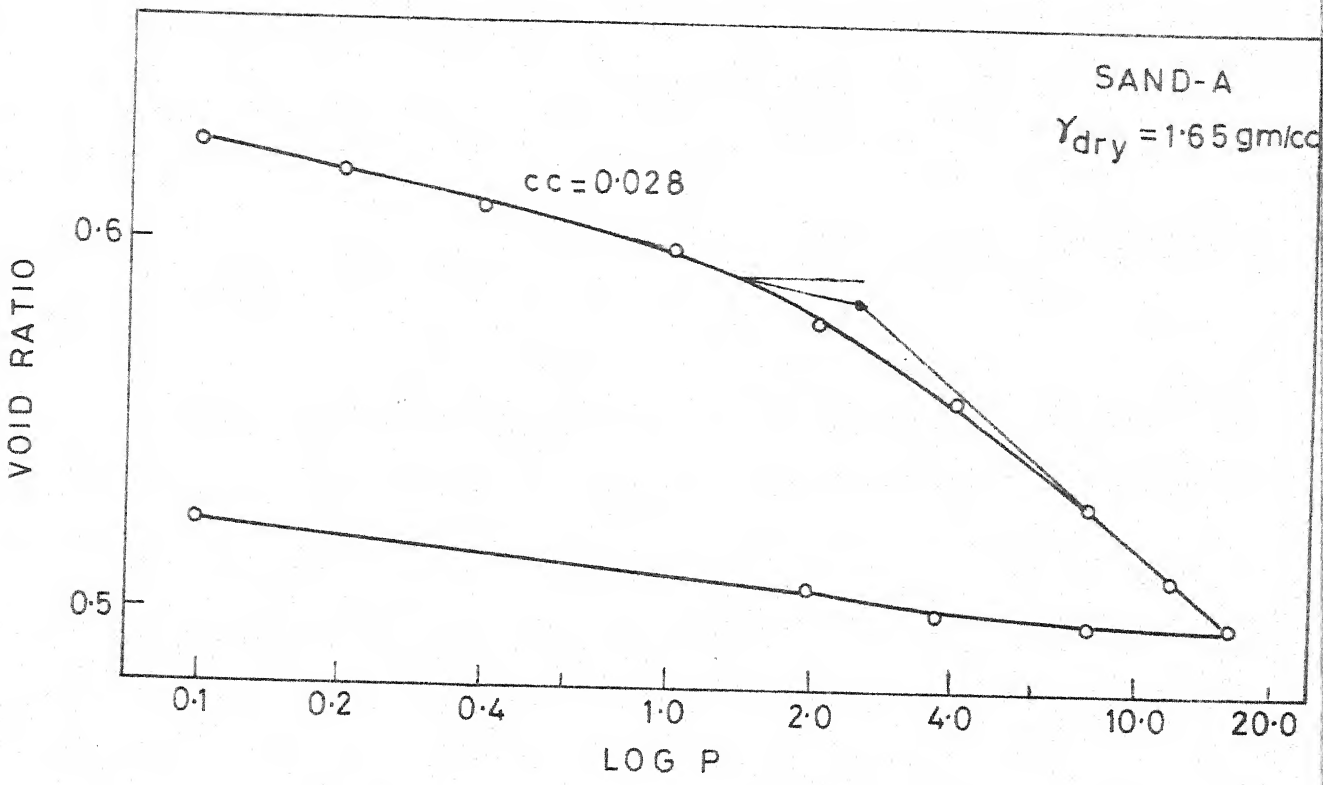


FIG.3·8 DIRECT SHEAR TEST RESULTS FOR SANDS A & B



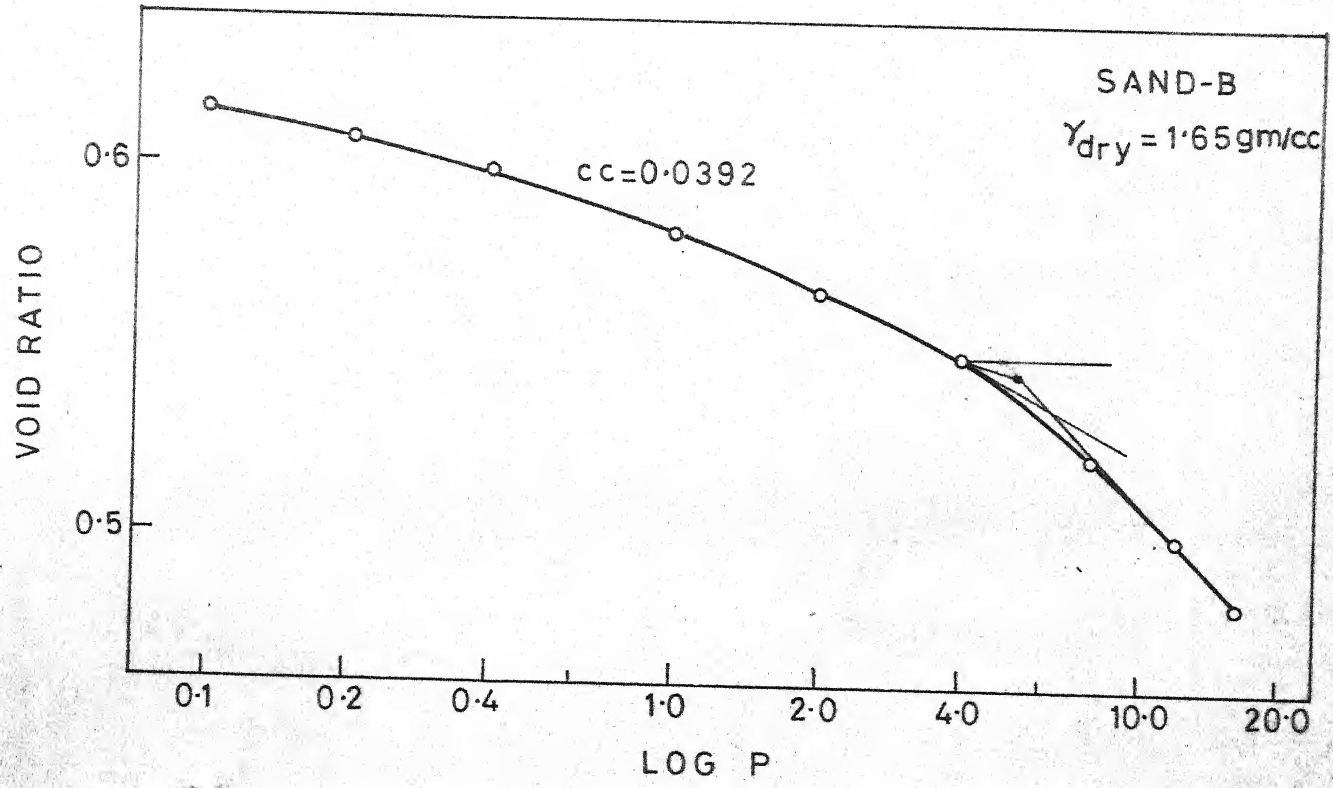


FIG.3·9 e-LOG (P) CURVES FOR SANDS A & B
(OEDO METER)

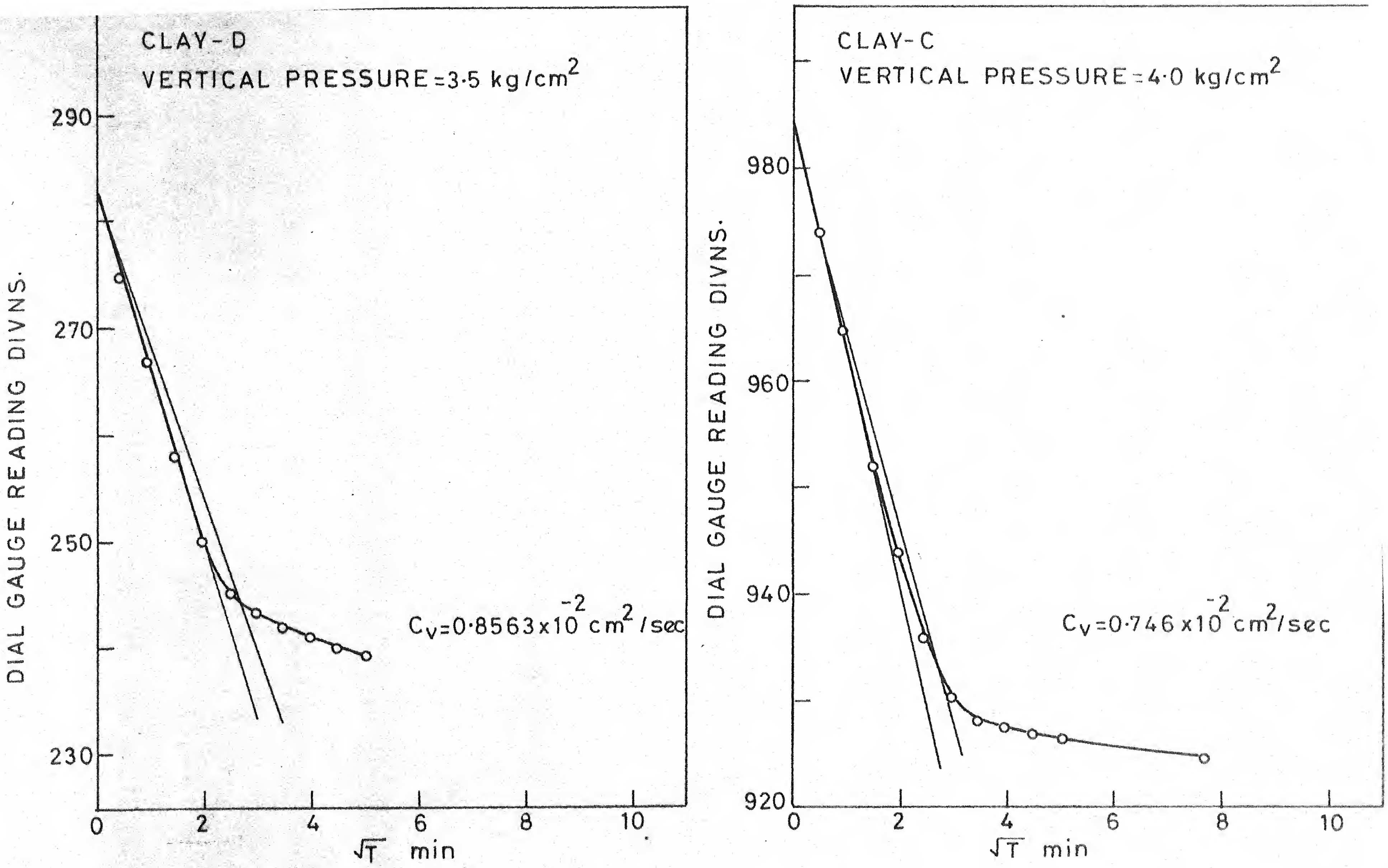


FIG.3·10 OEDOMETER TEST DATA FOR CLAYS C & D

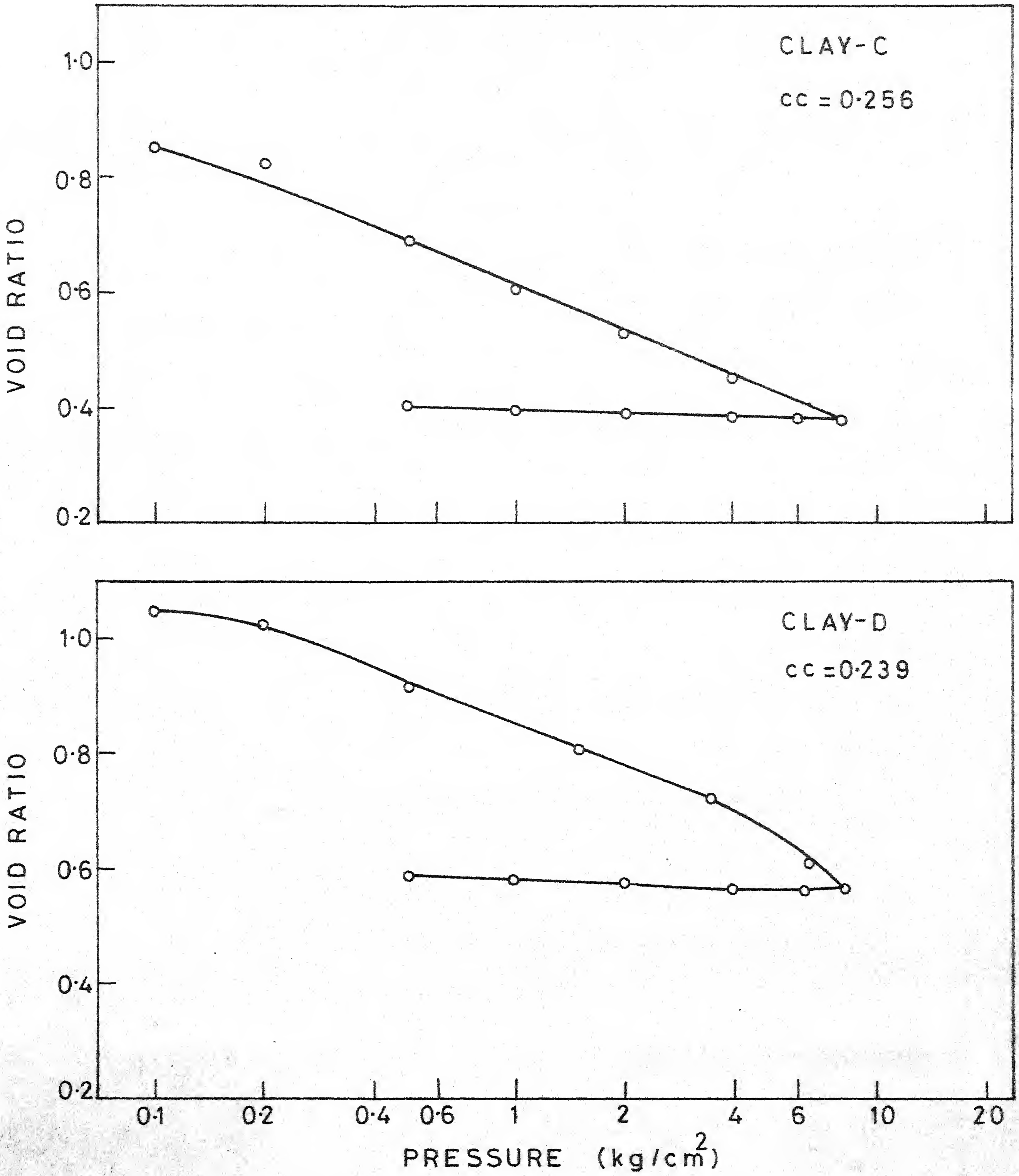


FIG. 3·11 e-Log P CURVES FOR CLAYS C & D

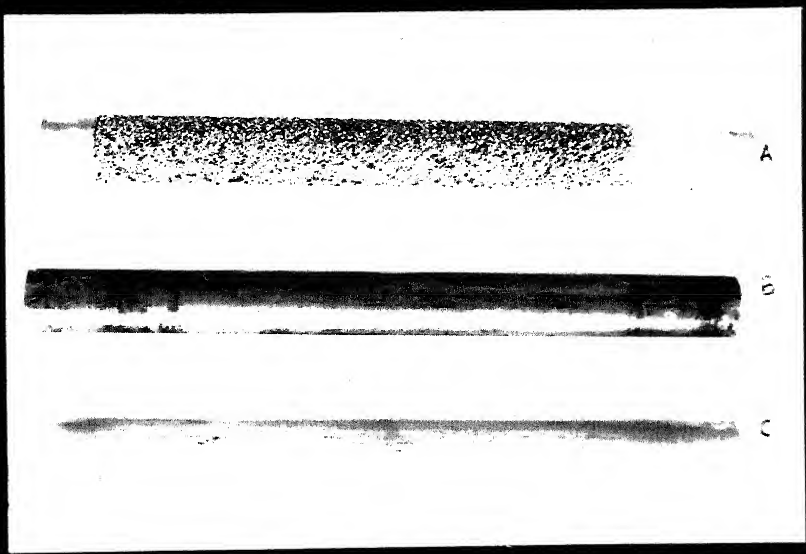


Plate No. 3.1 Types of Piles tested
A) Rough B) Brass C) Aluminum

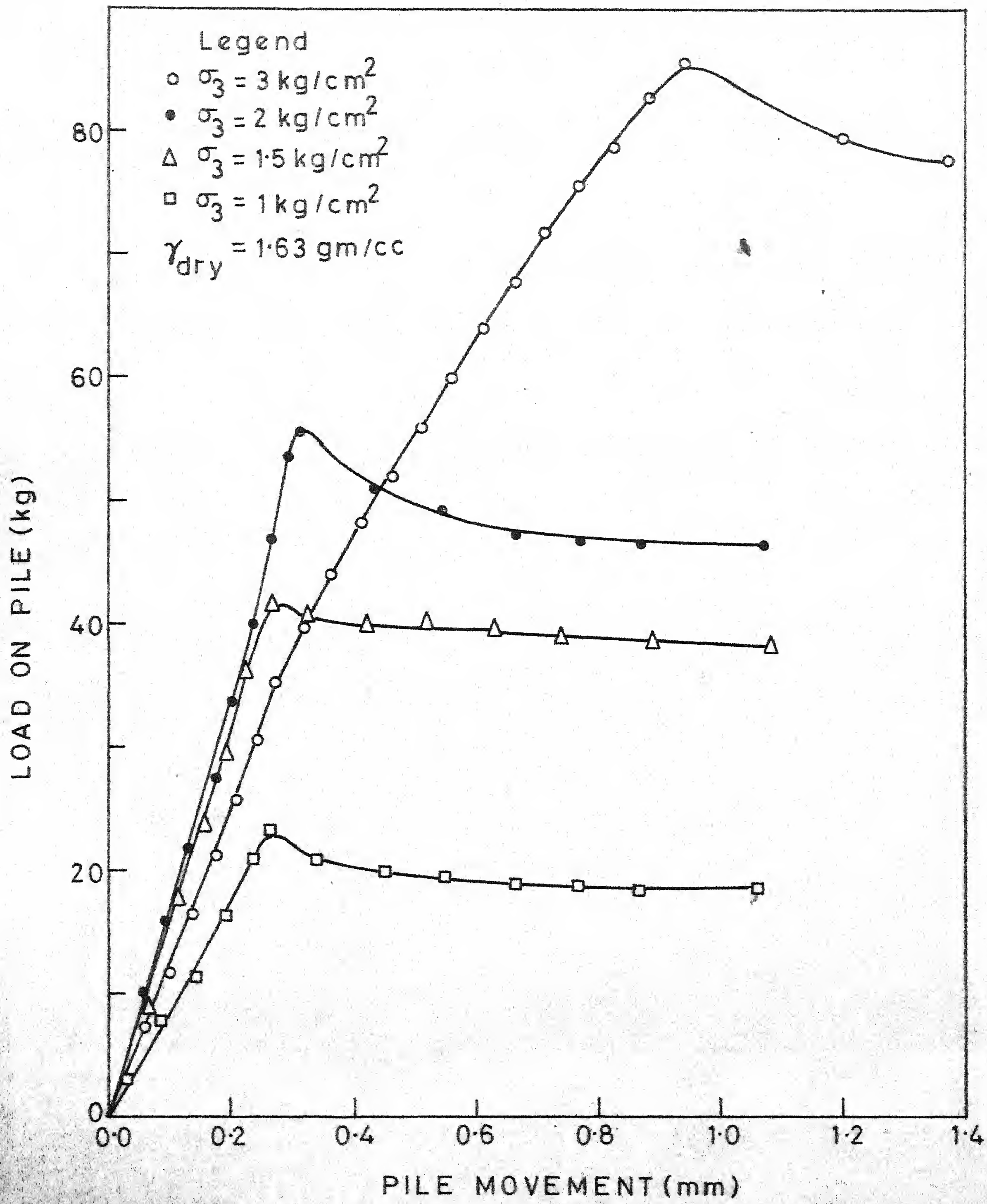


FIG. 3·12 LOAD-DISPLACEMENT CURVES FOR PILES IN SAND A (Aluminum pile)

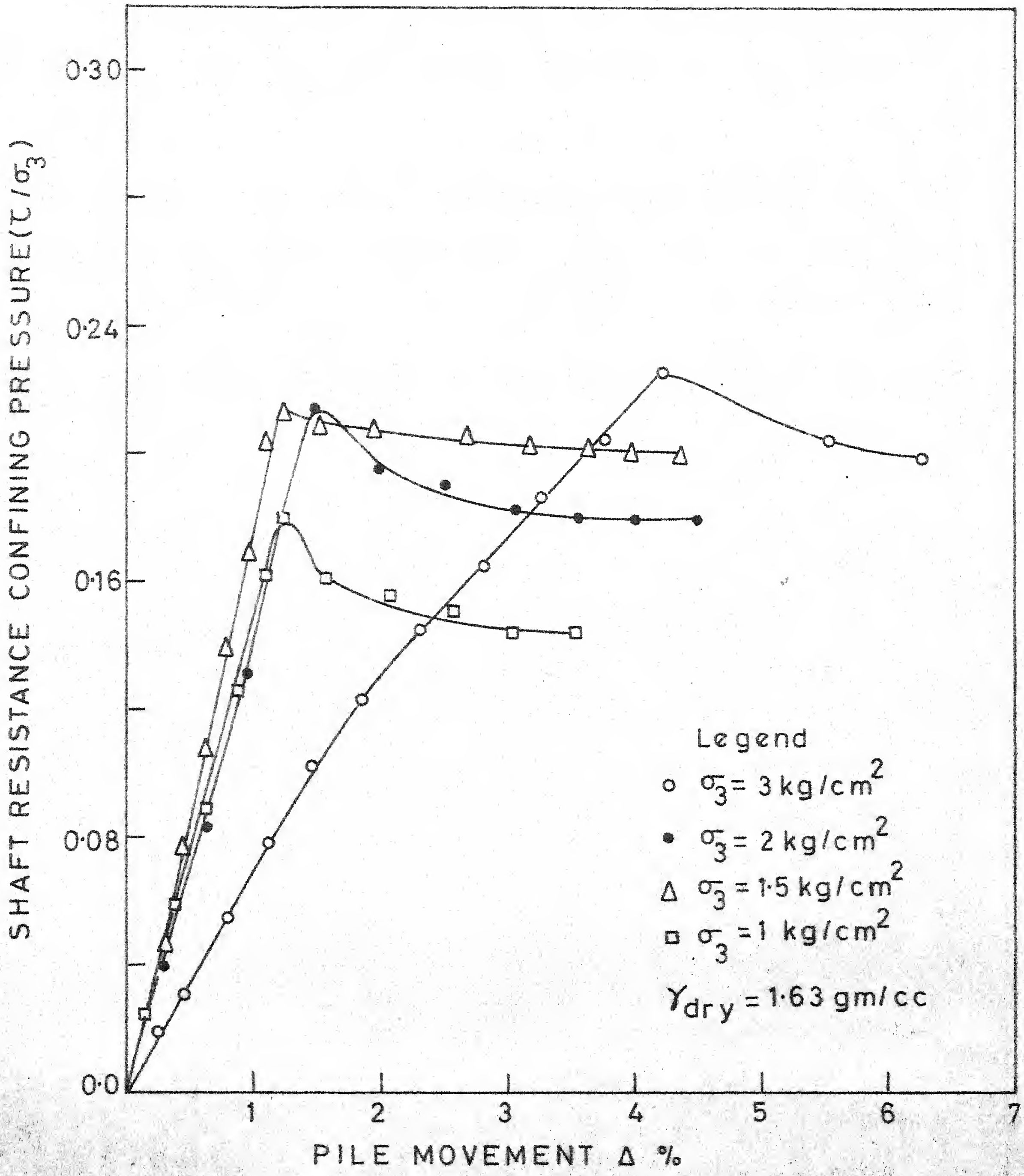


FIG. 3·13 NONDIMENSIONAL τ-Δ CURVES FOR PILE IN SAND A (Aluminum pile)

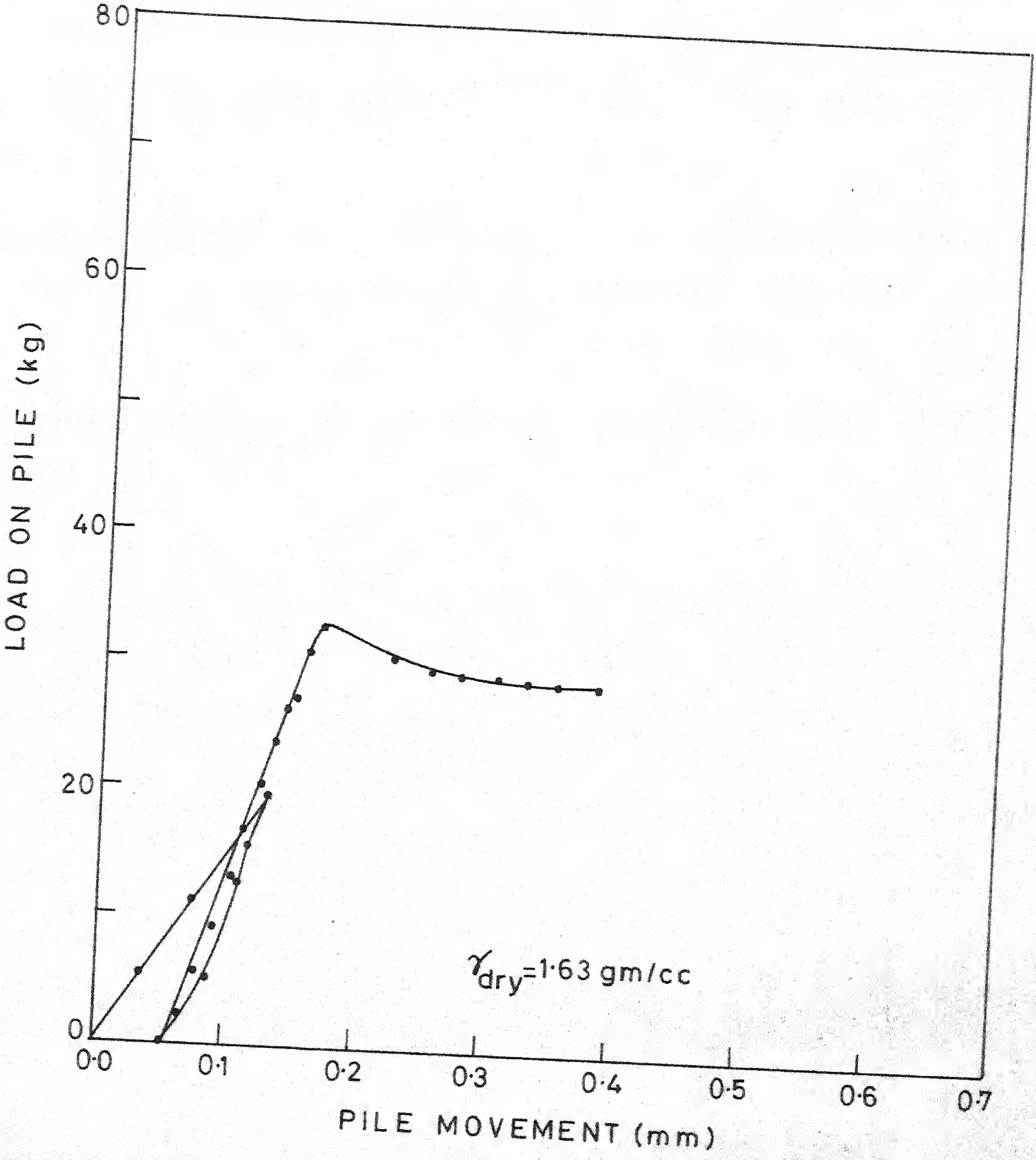


FIG. 3·14 CYCLIC LOAD TEST ON PILE IN SAND A (σ_3=1·0 kg/cm^2) (Aluminum pile)

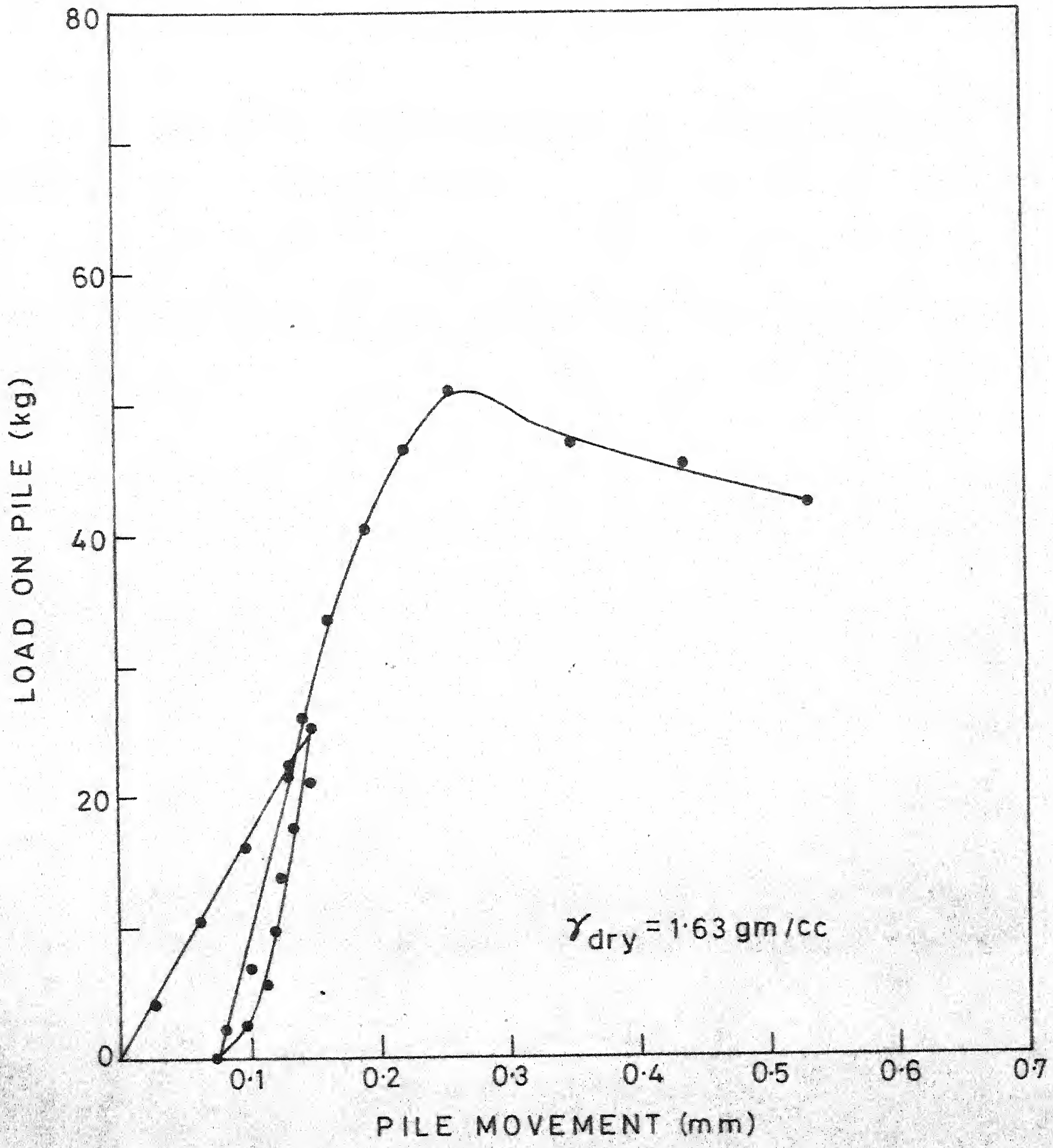


FIG. 3·15 CYCLIC LOAD TEST ON PILE IN SAND A (σ_3 = 1·5 kg/cm^2) (Aluminum pile)

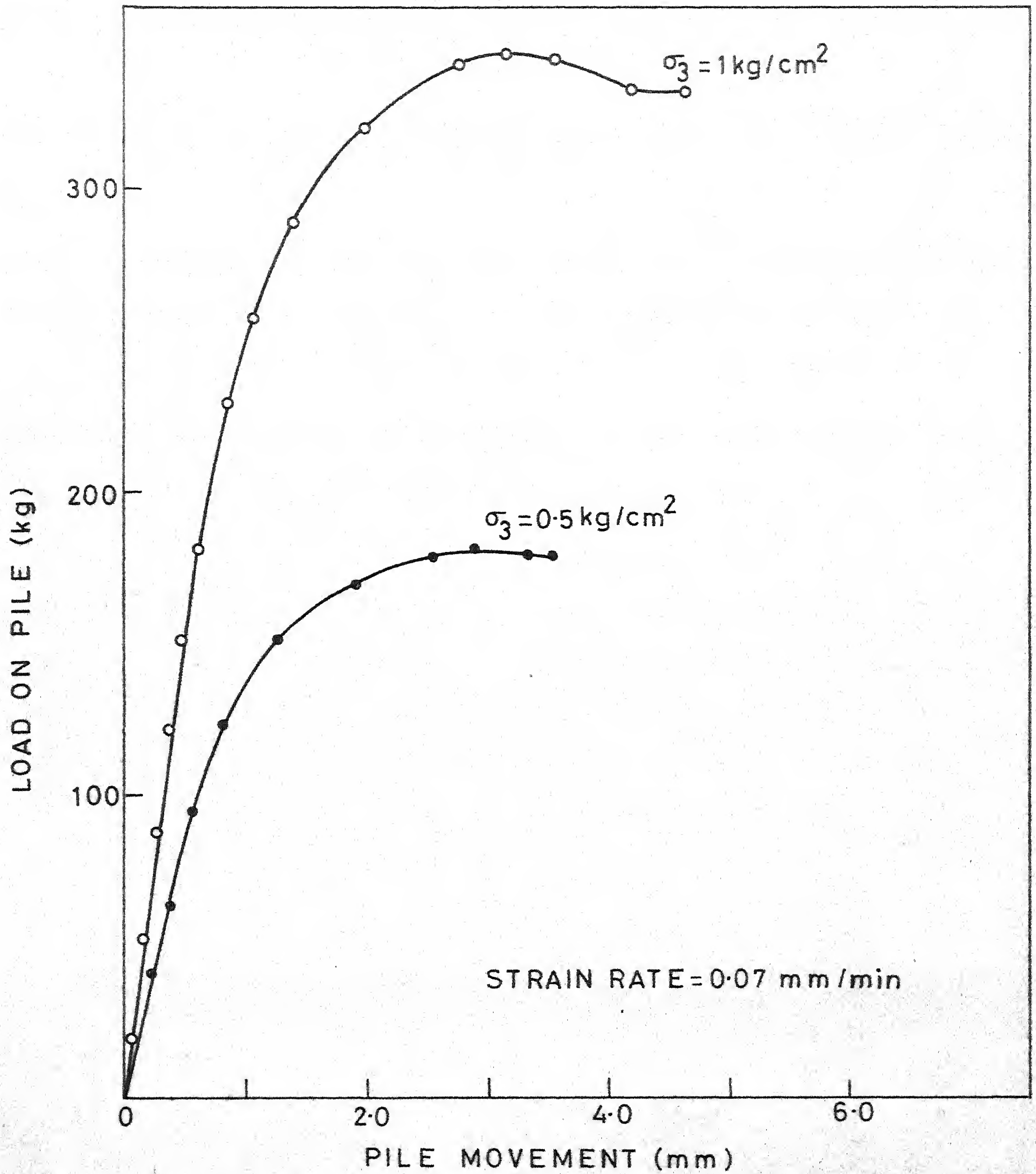


FIG. 3·16 LOAD-DISPLACEMENT CURVES FOR ROUGH PILE IN SAND A

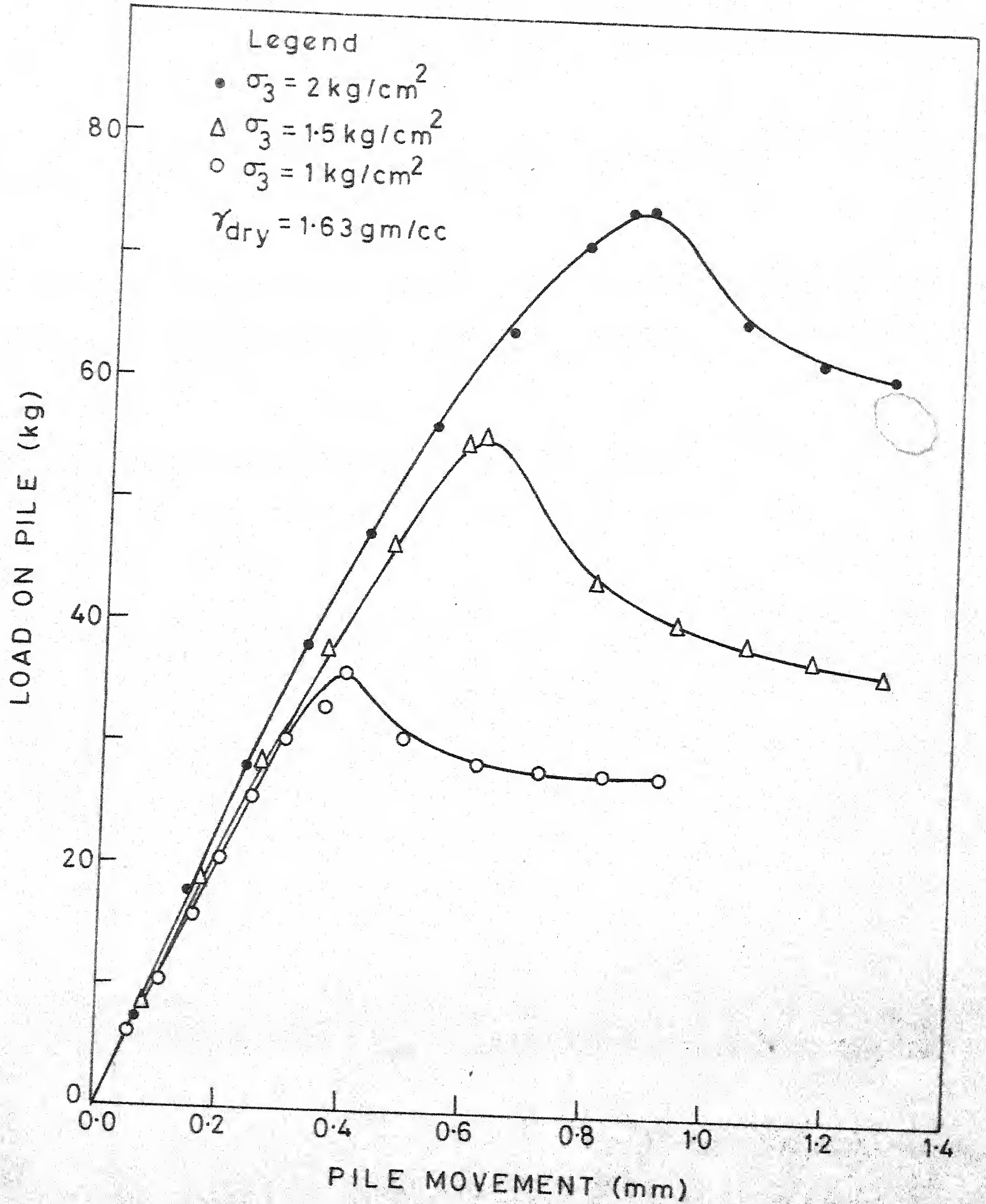


FIG. 3·17 LOAD-DISPLACEMENT CURVES FOR PILES IN SAND B (Aluminum pile)

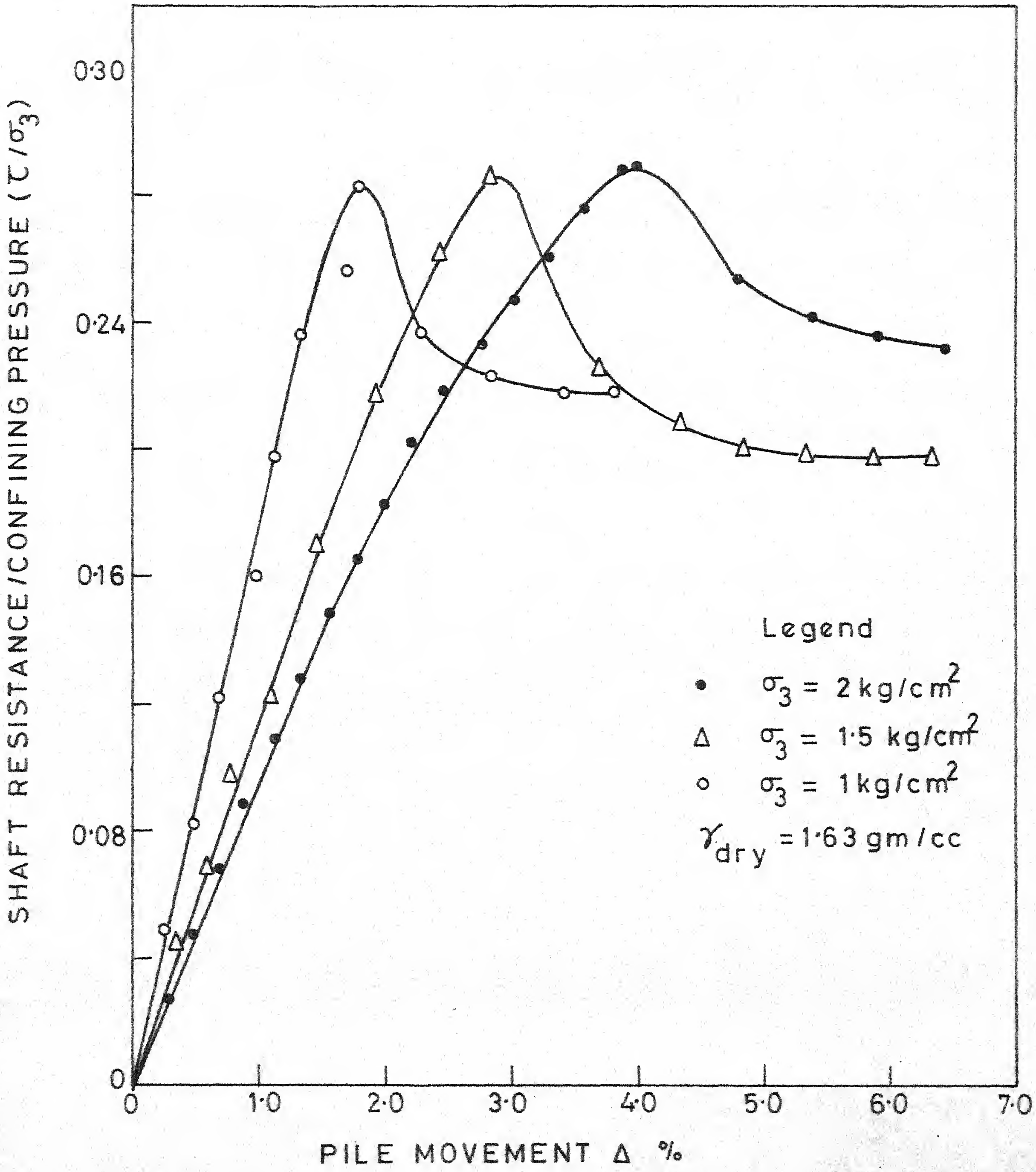


FIG.3.18 NONDIMENSIONAL τ - Δ CURVES FOR PILES IN SAND B (Aluminum pile)

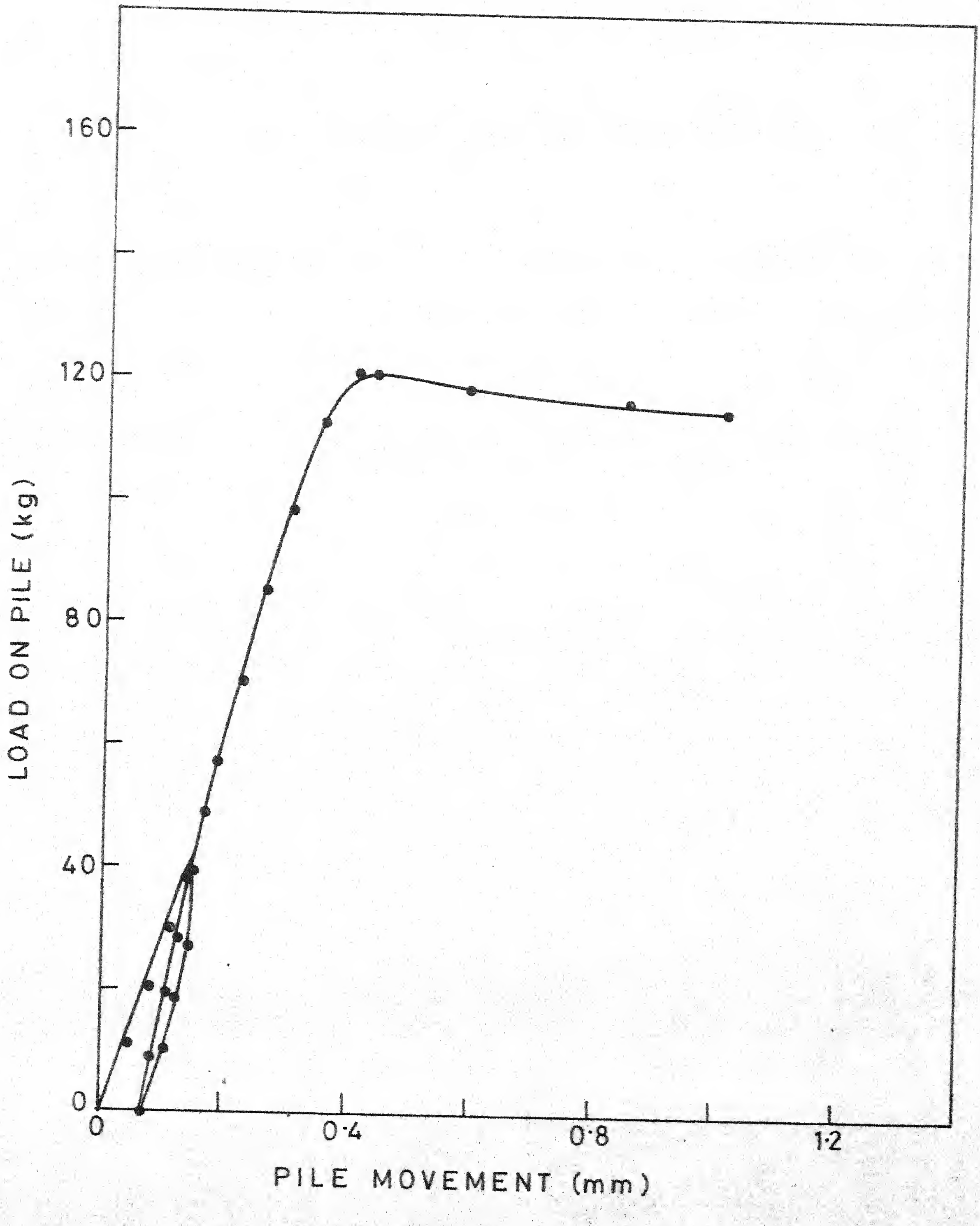


FIG. 3·19 CYCLIC LOAD TEST ON PILE IN SAND B ($\sigma_3 = 2\ kg/cm^2$)

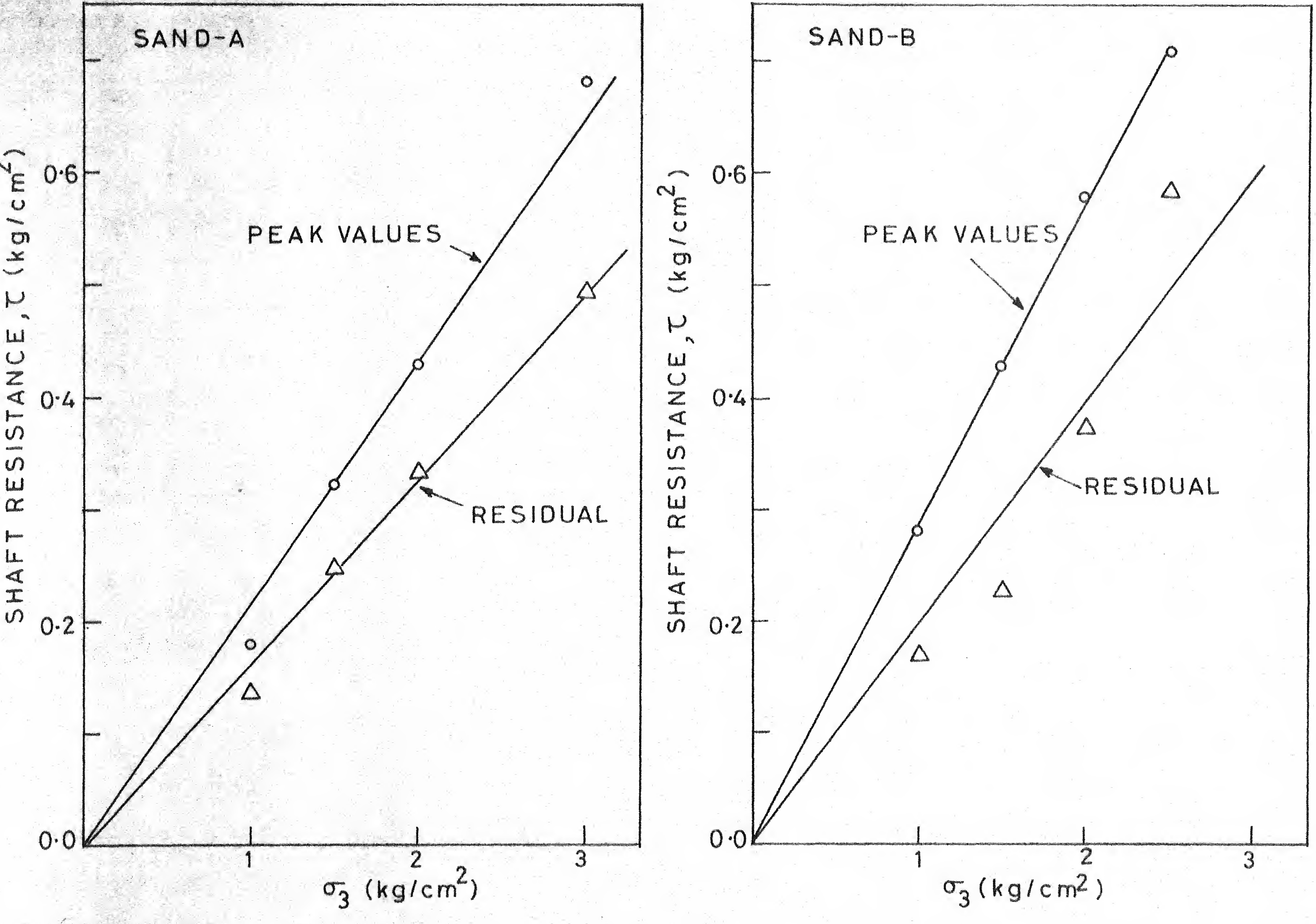


FIG.3·20 SUMMARY OF PILE-LOAD-TESTS ON SANDS A & B

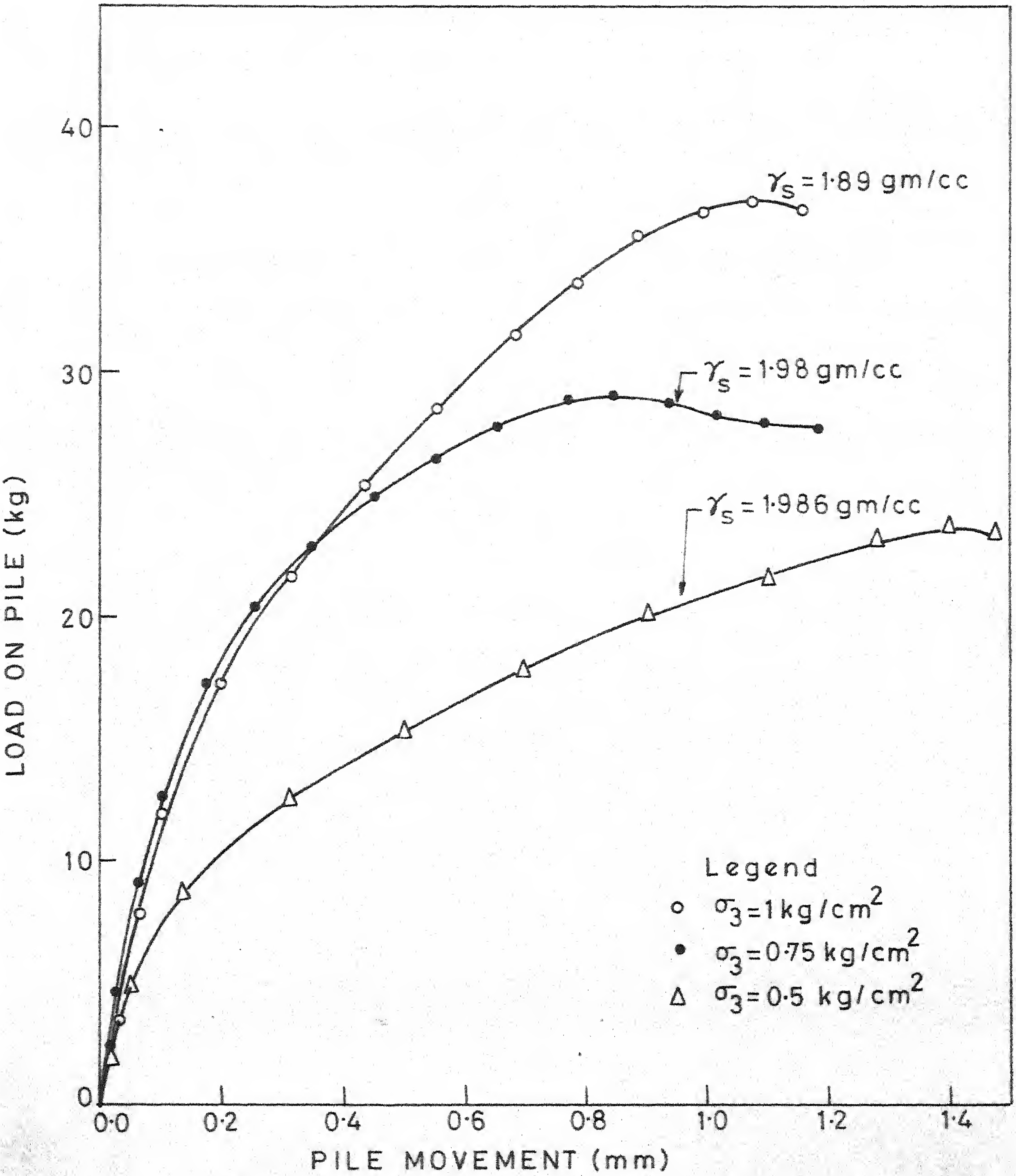
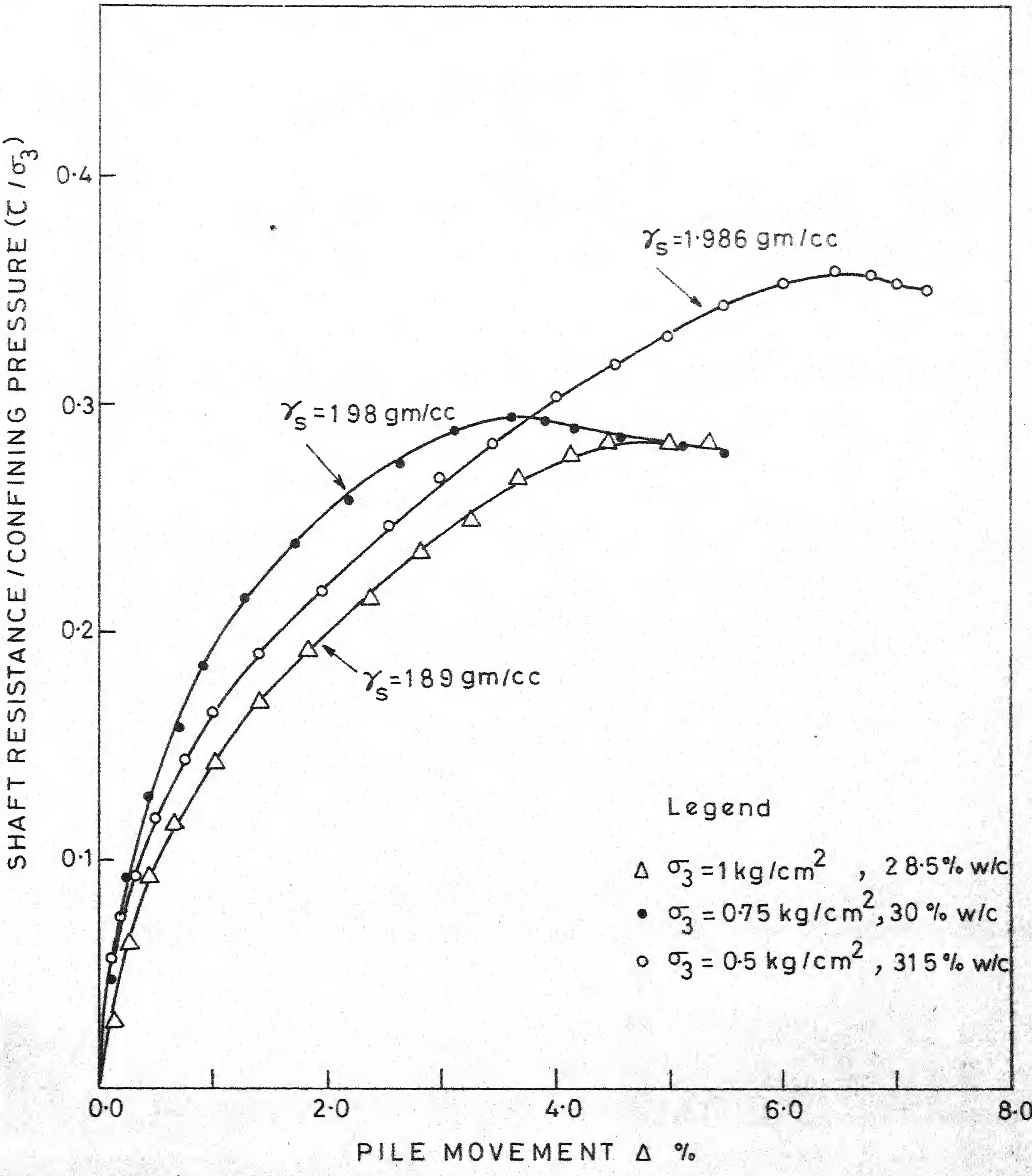


FIG. 3·21 LOAD-DISPLACEMENT CURVES FOR PILES IN CLAY C (Brass pile)



FIG·3·22 NONDIMENSIONAL $\tau - \Delta$ CURVES FOR PILES IN CLAY C (Brass pile)

6

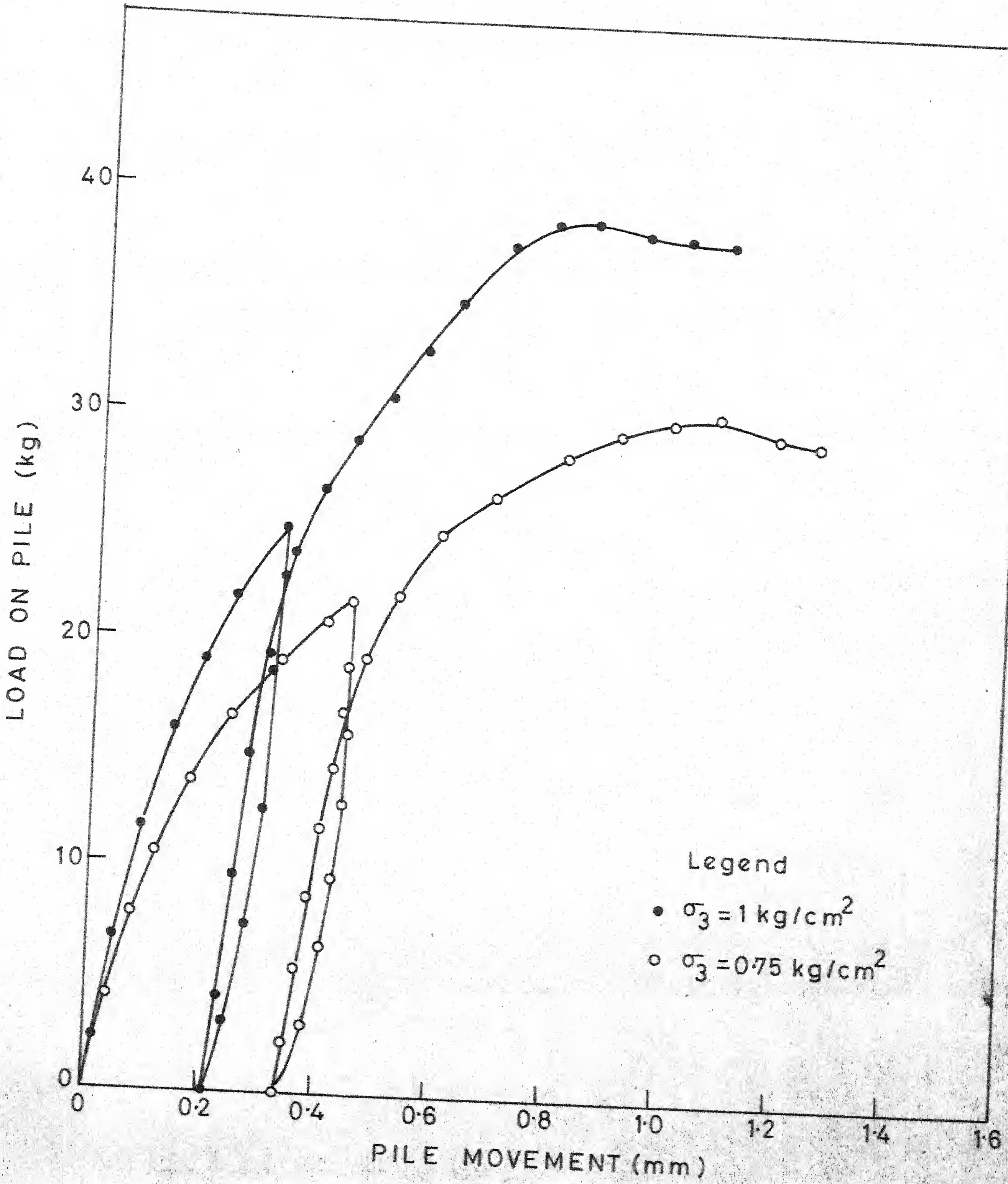


FIG. 3·23 CYCLIC LOAD TESTS ON PILE IN CLAY C
(Brass pile)

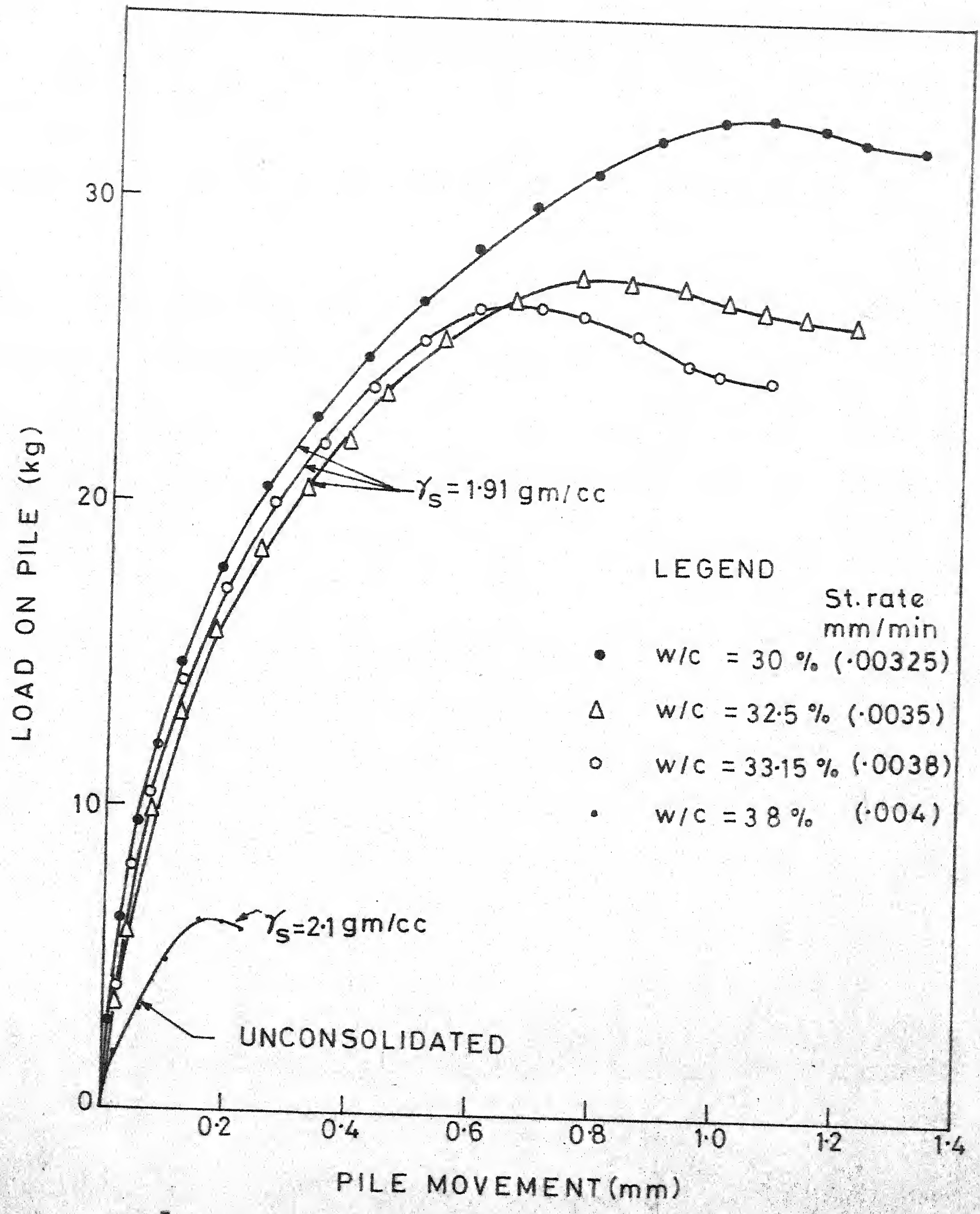


FIG. 3·24 LOAD-DISPLACEMENT CURVES FOR PILES IN CLAY D (σ_3=0·75 kg/cm^2) FOR DIFFERENT LOADING RATES (Brass pile)

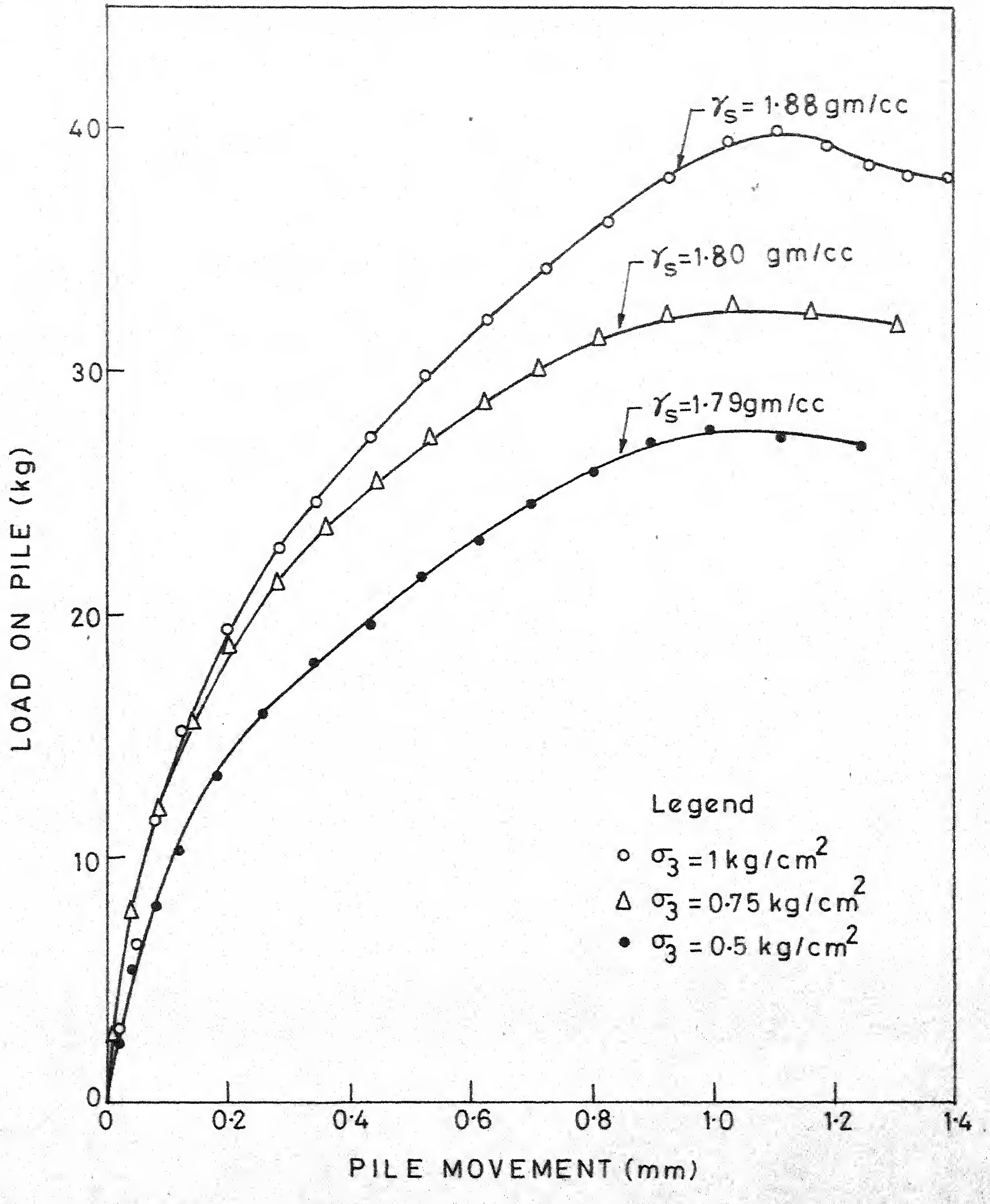


FIG. 3·25 LOAD-DISPLACEMENT CURVES FOR PILES IN CLAY D (Brass pile)

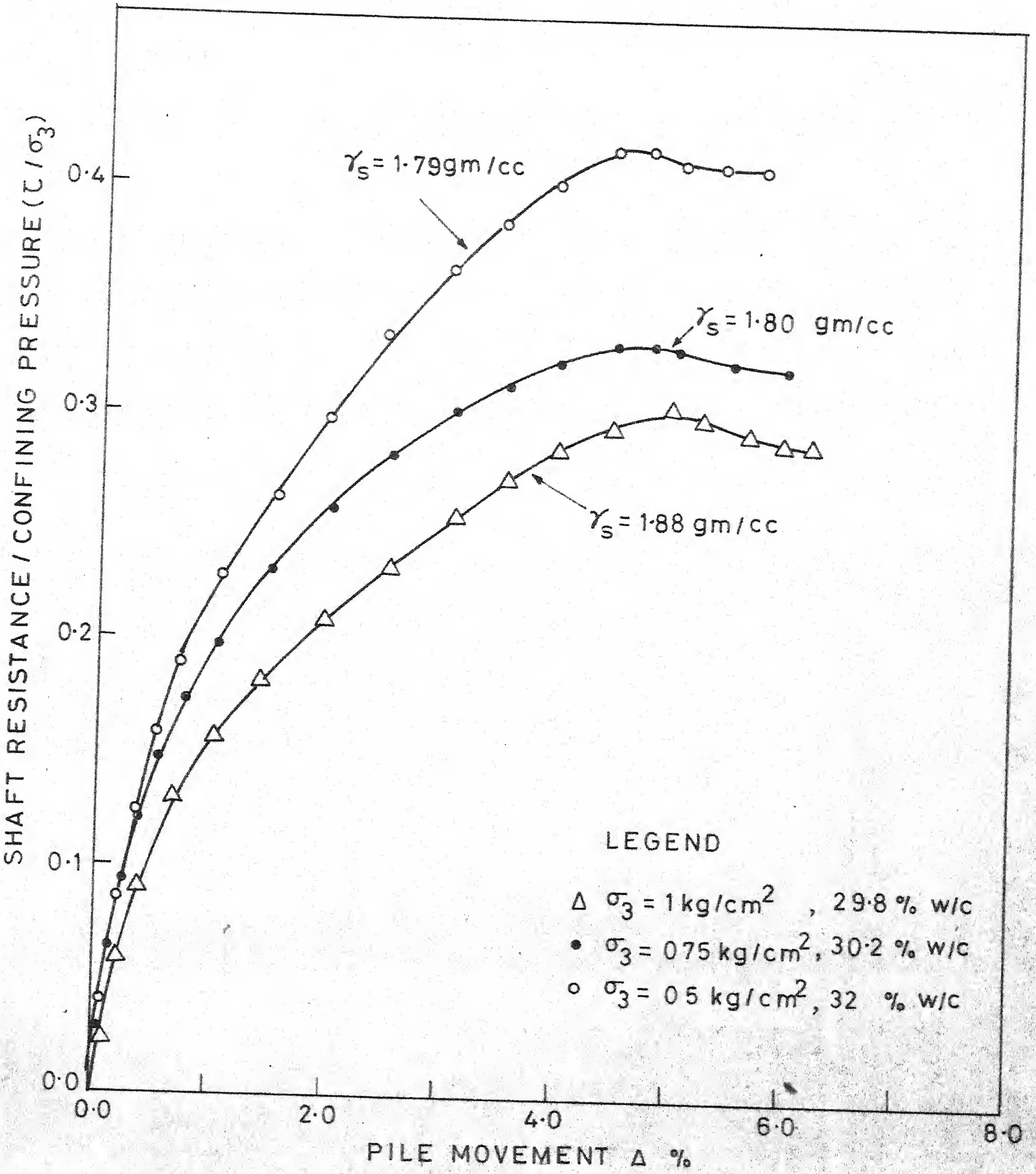
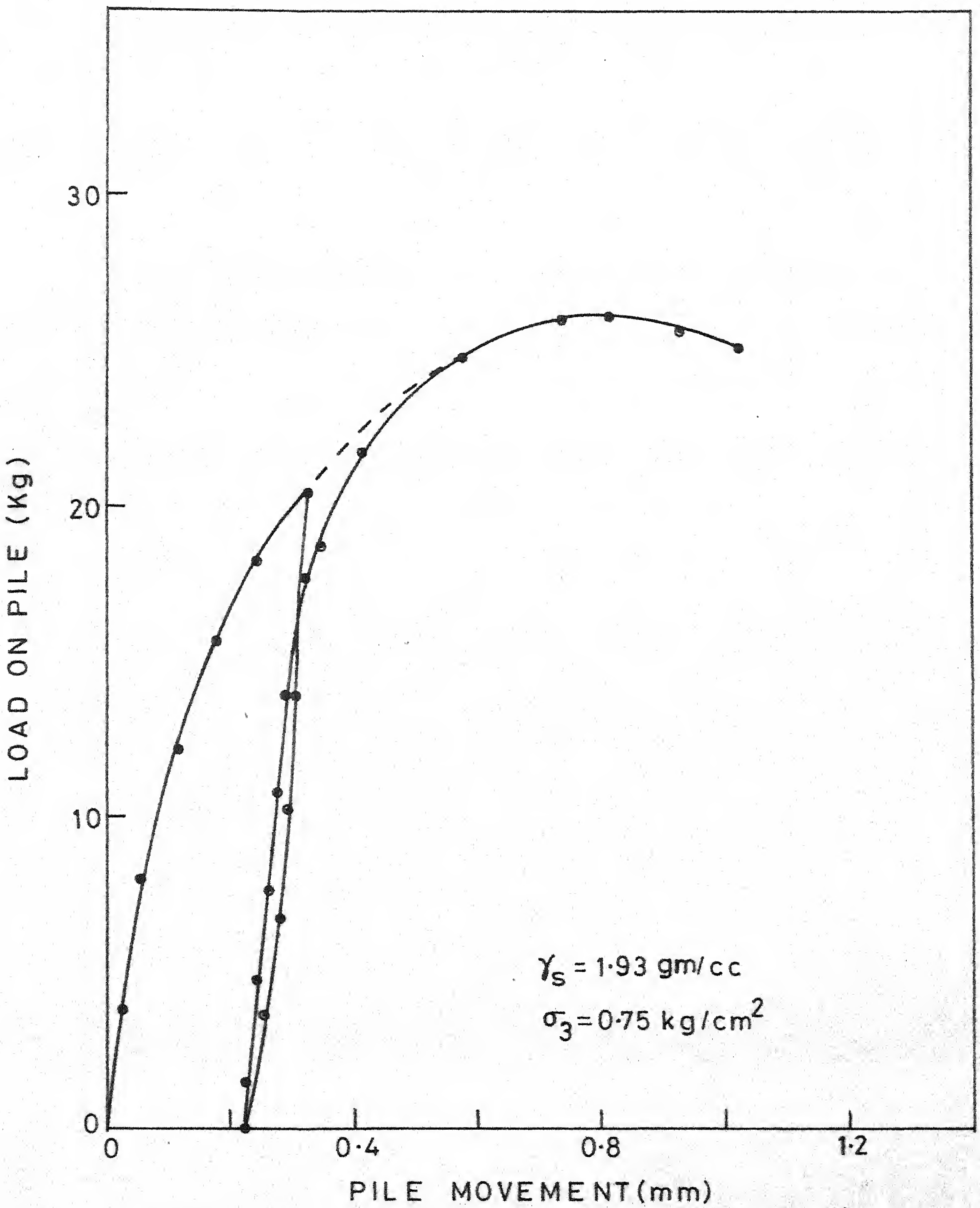


FIG. 3·26 NONDIMENSIONAL $\tau - \Delta$ CURVES FOR PILES IN CLAY D (Brass pile)



FIG·3·27 CYCLIC LOAD TESTS ON PILES IN CLAY D (σ_3 = 0·75 kg/cm^2) (Brass pile)

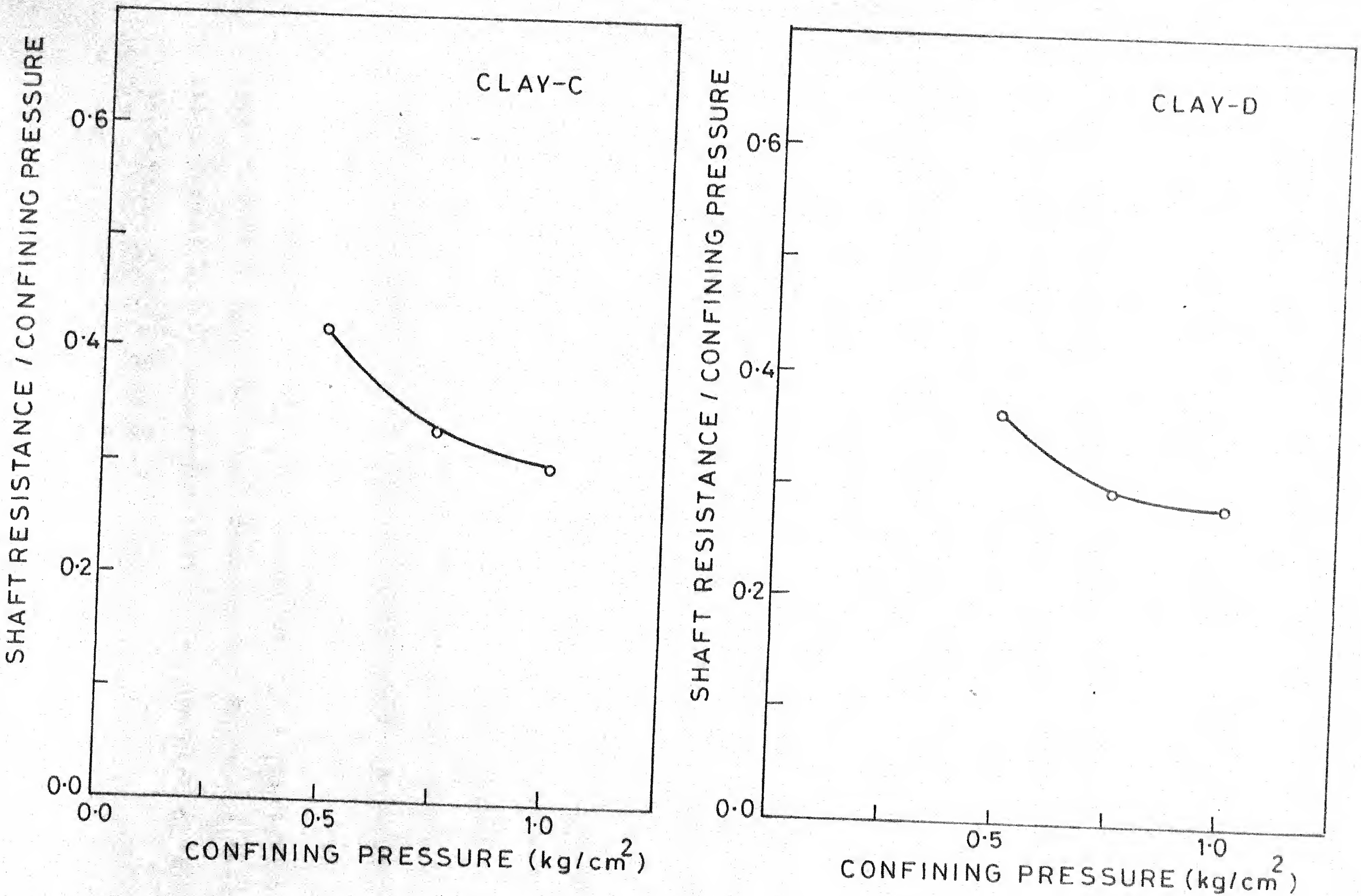


FIG. 3·28 SUMMARY OF PILE-LOAD-TESTS ON CLAYS C & D

CHAPTER 4

INTERPRETATION OF PILE LOAD TEST DATA

4.1 INTRODUCTION

Pile load tests are conducted for three main purposes as Fellenius (1975) mentioned :

1) As a part of research investigation. The results of the tests are to be used for analysing the pile behaviou and soil-pile interaction under loads.
2) As a part of a field investigation at a particular sitc prior to driving of the contract piles.
3) As a check on contract piles, in which case the contract pile itself is being test loaded.

For all the above objectives, the analysis of the test data with proper care and accuracy are needed to obtain reasonable conclusions. If the pile is test loaded for the first purpose mentioned above, in most of the cases the pile may be instrumented, which necessitate a large expenditure.

The splitting up of the total load at a particular pile movement into the load taken by the shaft and that transferred by the tip would form the major part of the interpretation of pile load test data.

The test-load data obtained from the load tests on instrumented piles may be useful in arriving at an easier and reasonable procedure for separating the shaft and end resistances. A semi-empirical approach is proposed as a method for the interpretation of pile load test data. A brief review of the earlier works and suggestions are given in this chapter. An analysis of Whitaker's (1966) pile load test data is carried out based on which the proposed procedure is developed.

4.2 METHODS AVAILABLE

The concept of the separate evaluation of the shaft resistance and the base resistance forms the basis of all calculations of pile capacity. The basic equation is

$$Q_u = Q_B + Q_s - W_p \tag{4.1}$$

where Q_u is the ultimate resistance of the pile, Q_b is the ultimate resistance of the base, Q_s is the ultimate resistance of the shaft and W_p is the weight of the pile. The term W_p is generally ignored as it is small in relation to Q_u to form an equation

$$Q_u = Q_b + Q_s \tag{4.2}$$

The interpretation of the pile test data consists of evaluating one of the factors on the RHS of the equation (4.2) from the test data, and the knowledge of physical

properties of the soil mass. Here the estimation of the second term, i.e., the resistance offered by the sides of the shaft is considered. Expressions for unit shaft friction (f_s) in cohesive soils based on undrained shear strength S_u have been widely used in the form :

$$f_s = \alpha \cdot S_u \tag{4.3}$$

where the range of the values for α may be from 0.3 to more than 1.0. An expression

$$Q_s = \frac{1}{2} K_s \, \sigma'_v \tan \delta \, A_s \tag{4.4}$$

is being generally used in the case of piles in cohesionless soil where Q_s is ultimate side friction, K_s is the average lateral earth pressure, σ'_v is the effective over burden pressure at pile base level, δ is the angle of friction between the pile and the soil and A_s is the area of the shaft in contact with the soil.

There are several modified relations for skin friction of a pile in cohesive and cohesionless soils, based on effective stress principle and standard test procedures using which predetermination of ultimate shaft resistance is possible. Some of the methods suggested earlier by others to estimate the shaft resistance from a pile load test data are reviewed here.

Van Weele (1957) suggested that, since the total skin friction remains the same after a certain settlement of the pile, any additional increase in load is taken by base of the pile. After analysing field data, he concluded that a line drawn parallel to the linear portion of the load settlement curve at larger settlements through the zero settlement point approximately divides the total load into shaft resistance and base resistance at any pile movement. To apply this method the pile should be loaded with significantly large loads. He considered the base resistance to increase linearly with the movement of the pile, which is not the case in general. It was suggested that from a load test upto failure, the modulus of subgrade reaction could be calculated and used to predict the base resistance for other piles in the same site.

Bazant and Masopust (1980) approach the problem in the same way and find it reasonable. Pull-out tests were suggested by several authors (Mohan and Chandran, 1961). When a pile is being pulled out, the total resistance offered by the pile is entirely due to the shaft. But most of the pull-out-tests gave a smaller value of shaft resistance than that offered when the pile is under compressive loads.

The cyclic load test is particularly useful for separating the load carried by the pile into shaft resistance and base resistance (Mohan and Chandran, 1961). The elastic compression ΔL of the pile corresponding to any load Q can be calculated from the following expression based on Hooke's law

$$\Delta L = [Q - Q_s/2]L/AE \qquad (4.5)$$

where Q_s is that part of load taken by sides of the shaft. A is gross area of the shaft, E is the elastic modulus of the pile and L is the length of the shaft. Based on the experimental findings of Van Weele (1957) as mentioned, the total load Q at any stage can be separated into shaft Q_s, and base Q_B resistances; with the understanding that Q_B is proportional to pile movements.

Since Q_s is not known to start with, the elastic compression of the pile is assumed to be zero, and hence the elastic compression of the soil S_s equal to the total elastic recovery of the pile top obtained from cyclic load test data.

Plot a curve showing the relationship between Q and corresponding elastic recoveries obtained from test data.

Using the principle of Van Weele, get the shaft resistances at each loading, and hence the elastic compression of the pile using the relation (4.5).

Having obtained the elastic compression of the soil S_s, by subtracting the elastic compression of the pile, plot a new curve with new values of S_s and corresponding Q.

Repeat the procedure of finding out Q_s, and hence ΔL and S_s till a curve which gives a sufficiently accurate values for Q_s and Q_B. It has been observed that the third trial of curves gives sufficiently accurate results.

Chin (1977) suggested that in the case of piles embedded in soil for which the mobilisation of base resistance requires relatively large displacements as compared with that for the mobilisation of shaft friction, the initial load acting on the pile will be borne almost entirely by the shaft friction.

Load test data is replotted as Δ/P against an abscissa of where Δ is the pile movement corresponding to a load P. He suggests that the initial part of the plot, consists of a straightline, the inverse slope of which gives the ultimate value of shaft resistance in compression, whereas the inverse slope of the second part of the curve gives the ultimate total load on the pile. He analysed the data furnished by Webb (1977), Thoybern and Macvical (1970) to see the procedure works.

Promboon and Brenuer (1980) tried the procedure proposed by Chin for large bored piles in Bangkok clay and concluded that the ultimate load of a bored pile in clay can be predicted easily within ± 10 percent accuracy, from the hyperbolic method. But, the Δ/P V/s Δ curves did not give the shaft and base resistances separately.

4.3 ANALYSIS OF WHITAKER'S DATA

Data of pile loading tests from instrumental deep bored cylindrical piles in London clay furnished by Whitaker and, Cooke (1966) were analysed to suggest a procedure for separating the shaft resistance and the base resistance from the overall Load-Settlement curves. Test data on instrumented piles (D,H,k,N,E,L,M and P) were selected for the comparison of the estimated shaft resistances with the observed ones.

An attempt to analyse the total load - settlement response by considering the hyperbolic relation as suggested by Chin (1977) was made. The analysis of Whitakers data by this procedure revealed that the inverse slope from the maintained load test data does not agree with the estimates of the ultimate total resistances. The suggestion on separating out base and shaft resistances is also not found to be reliable for these data. The test data and the corresponding hyperbolic plots are presented in Figs. 4.1 to 4.8 for the piles studied.

A theoretical generation of the total load - displacement curves in attempted, considering the base resistance Vs displacement (q_B Vs Δ) and shaft resistance vs displacement (q_S vs Δ) as hyperbolae. The total resistance of a pile can be expressed as

$$Q = \frac{\Delta}{a_s + b_s \Delta} + \frac{\Delta}{a_b + b_b \Delta} \tag{4.6}$$

where a_s and a_b are inverse of the initial tangents of q_S vs Δ and q_B vs Δ curves respectively, and b_S and b_B are the inverse of the ultimate shaft and base resistances.

The above expression can be rewritten as

$$Q/A = \left[\frac{1}{\frac{(1+\alpha)}{K} + \frac{(1+\beta)}{Q_u}\Delta} + \frac{1}{\frac{(1+\alpha)}{\alpha K} + \frac{(1+\beta)}{\beta Q_u}\Delta}\right] \tag{4.7}$$

where

$\alpha = a_b/a_S$

$\beta = b_b/b_S$

K = the intial slope of total load-displacement curve which is equal to $\frac{1}{a_b} + \frac{1}{a_S}$ and

Q_u = the ultimate total resistance which is equal to $\frac{1}{b_b} + \frac{1}{b_S}$

Reasonable values for K and Q_u are assumed and for different values of α from 1 to 8 load-displacement curves and corresponding hyperbolic curves are plotted for a constant

value of 1 for β (Fig. 4.9). For different values of β, for the same K and Q_u values, total load-displacement curves and corresponding hyperbolic curves are given in Fig. 4.10 for α equal to 4. It is interesting to note that the inverse slopes, do not give the ultimate resistances except in two cases - $\alpha = 1$ with $\beta = 1$ and $\alpha = 8$ with $\beta = 4$. But the separation of the total load into shaft and base resistance is not possible using inverse slope method of Chin (1977).

The data are analysed using the elastic solutions for movement of floating piles given by Poulos and Davis (1980). The base resistance - base displacement relationship is considered to be hyperbolic, and the shaft resistance - displacement relationship as elasto-plastic. The idealized curves are shown in Fig. 4.11.

It is considered that the initial tangent K of the load settlement curve is representative of the total effect of the base resistance and the shaft resistance. The soil modulus E_s is computed from the relation given by Poulose and Davis.

$$\rho = \frac{P.I}{E_s d} \qquad (4.6)$$

for floating piles where $I = I_o R_k R_h R_\nu$

ρ - settlement of pile head

P - applied axial load

I_o - settlement influence factor for incompressible in semi-infinite mass for $\nu_s = 0.5$

R_k - correction factor for pile compressibility

R_h - correction factor for finite depth of layer on a rigid base

R_ν - correction for Poisson's ratio

d - base diameter of the shaft

The values for R_k, R_L, $R\nu$ and I_o are given by Poulos and Davis (1980). The modulus of the soil is obtained from the initial tangent of the total load settlement curve from

$$E_s = \frac{K.I.}{d} \tag{4.7}$$

This value of E_s is used to determine the modulus of subgrade reaction, K_2, at the pile base, assuming the value of soil modulus at the level of the pile base to be same as the average soil modulus obtained before. To determine the value of K_2, the pile base is considered as a rigid loaded area embedded at a depth L, for which elastic solutions given by Poulose and Davis (1974) are valid.

$$I = P/2G\ a\ \delta_Z \tag{4.8}$$

where I is displacement factor, P/δ_Z modulus of subgrade reaction, G, shear modulus $= \frac{E_s}{2(1+\nu)}$ and a is the radius of the rigid area.

K_2, the initial tangent of the base load-base settlement curve is assumed to be dependent on the L/D ratio. As K is the total effect of the shaft and base stiffness, the slope

of the linear part of the shaft resistances may be obtained from :

$$K_1 = K - K_2 \tag{4.9}$$

Many experimental studies concluded that the shaft resistance is fully mobilized at relatively smaller pile movements, S_o, value of which depend mainly on the diameter of the shaft and the soil type, but independent of the pile length.

Values for K_2/K are computed for incompressible floating piles using the equations (4.1) and (4.3) (Poulos and Davis 1974, 1980) as a function of L/D ratio, ratio of base diameter to shaft diameter, d_b/d and ν of the soil. Fig. 4.12 gives K_2/K values for various values of L/D ratio, d_b/d ratio and . The intermediate values may be interpolated. If we know the value S_o and the ratio K_2/K it is possible to separate the total ultimate load into the shaft and base resistances using the relations

$$Q_S = K_1 \; S_o \tag{4.10}$$

and

$$Q_B = Q - Q_S \tag{4.11}$$

Whitaker's data was studied in that light to find the nature of the relation that exists between pile diameter and the movement at which the shaft resistance is mobilized. The values of S_o were back figured from the observed ultimate shaft resistance in equation (4.10). It is observed, from

Table 4.1 that for S_o an average value of 0.367 (percent of pile diameter) can be used for London clay. According to the average S_o, ultimate shaft resistances are computed and presented in the same table. Observed values of shaft resistances and that derived from inverse slope method are also included in Table 4.1 for comparison.

4.4 PROPOSED METHOD FOR ESTIMATING SHAFT RESISTANCE OF PILES IN LONDON CLAY

The procedure is explained in the previous section. The steps involved in the method are given below.

1. Plot the total load-settlement curve and determine the initial tangent K.
2. Using equation (4.7) compute the value of E_s and hence the value of G_s. The design curves furnished by Poulos and Davis (1980) could be used accordingly.
3. Compute the value for K_2 from the relation (4.8), and K_1 from $K_1 = K - K_2$ or by using the curves in Fig. 4.12.
4. The ultimate shaft resistance is computed using the relation $Q_s = K_1 S_o$.
5. The difference between observed total ultimate resistance and the ultimate shaft resistance gives the ultimate base resistance.

4.5 CONCLUSIONS

1. Inverse slope method proposed by Chin (1972, 78) did not give a reasonable estimation of the ultimate shaft and base resistance for the piles in London clay.

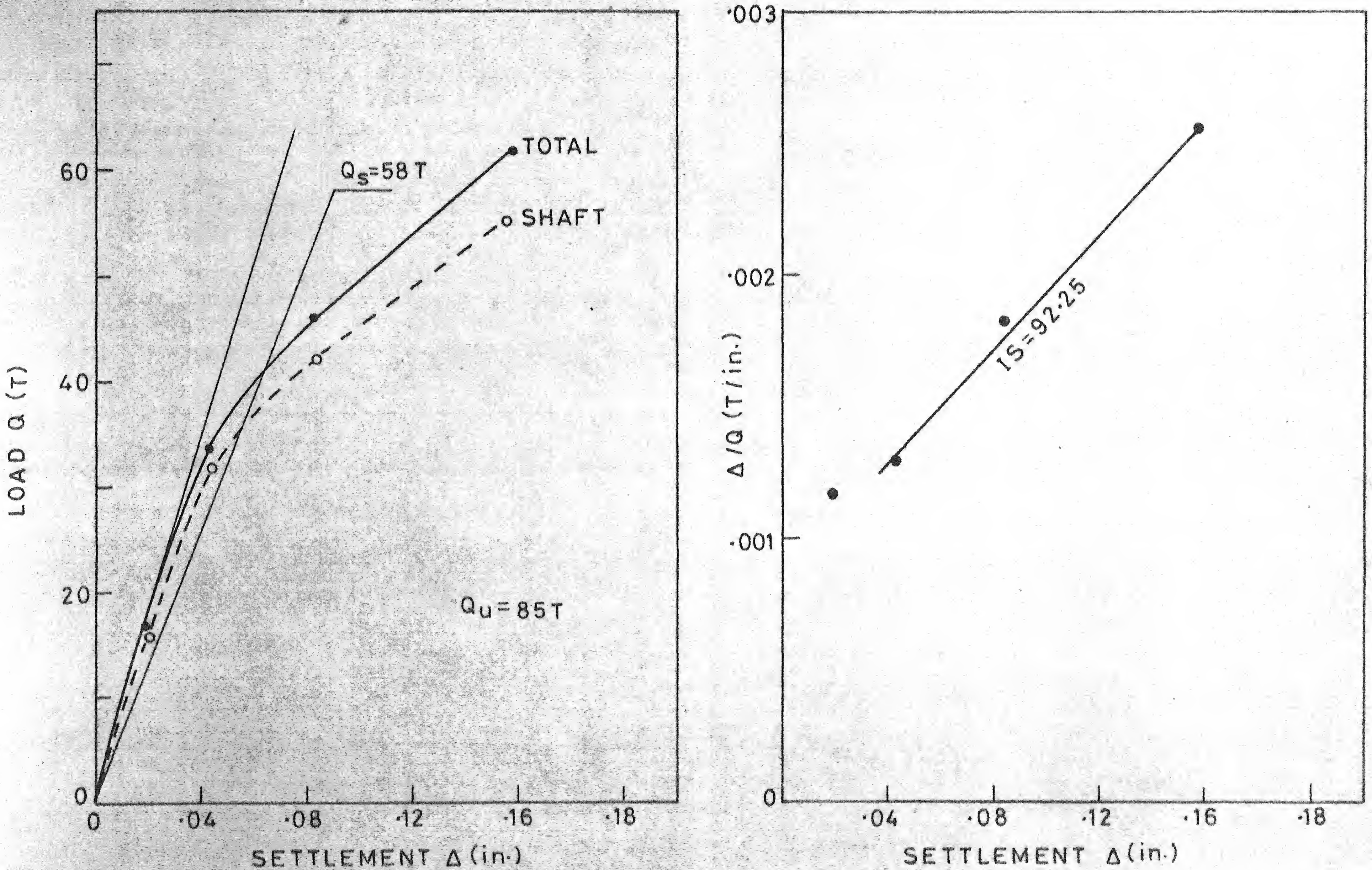
2. A new semi-empirical method is proposed.

Table 4.1 Analysis of Whitakers Data

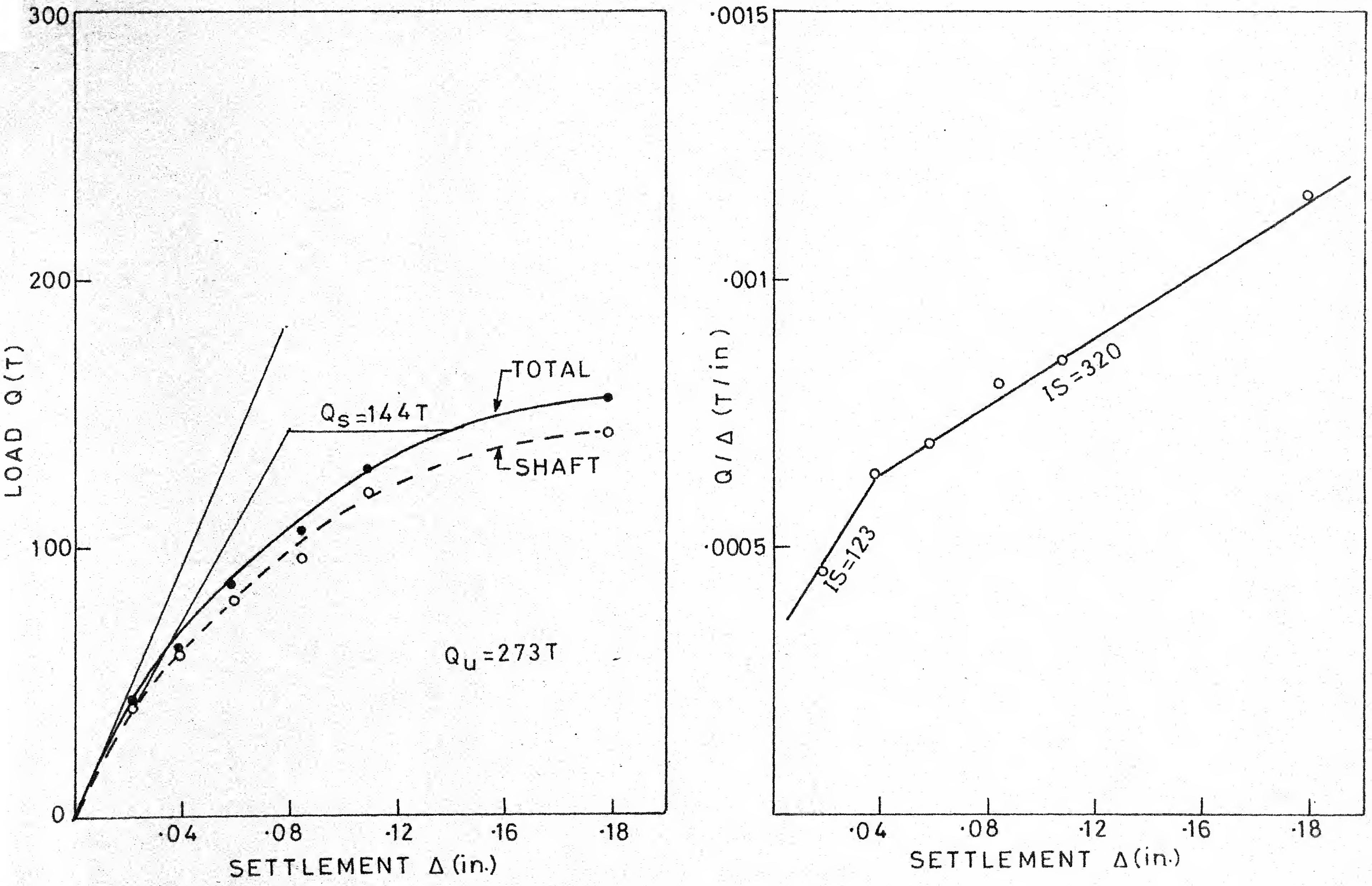
Pile	Shaft dia inches	Initial tangent modulus K, T/inc.	Computed E_s T/inch^2	Computed K_2 T/inch	$K_1 = (K-K_2)$ T/inch	Q_b (Whitaker)	IS of 1st st. line**	S_o of pile dia	Q_s* T	Ratio $\frac{\text{estimated}}{\text{observed}}$
D	25.0	854	3.67	217	637	55	-	0.372	58	1.055
H	30.5	2330	7.26	524	1806	144	123	0.2623	202	1.403
K	31.5	2000	5.28	394	1606	195	-	0.3857	186	0.954
N	37.0	2727	6.84	598	2129	276	421	0.3503	289	1.05
E	24.75	1640	5.90	669	971	87	392	0.3718	88	1.01
L	30.5	1565	5.23	816	749	77	232	0.3384	83	1.078
M	30.5	2240	5.32	830	1410	159	600	0.372	158	0.994
P	37.0	2540	5.73	990	1550	217	577	0.3784	210	0.968

*Using proposed method

** Using inverse slope method (IS), Chin (1972,77)



FIG·4·1 LOAD–DISPLACEMENT CURVE FOR PILE D



FIG·4·2 LOAD-DISPLACEMENT CURVE FOR PILE H

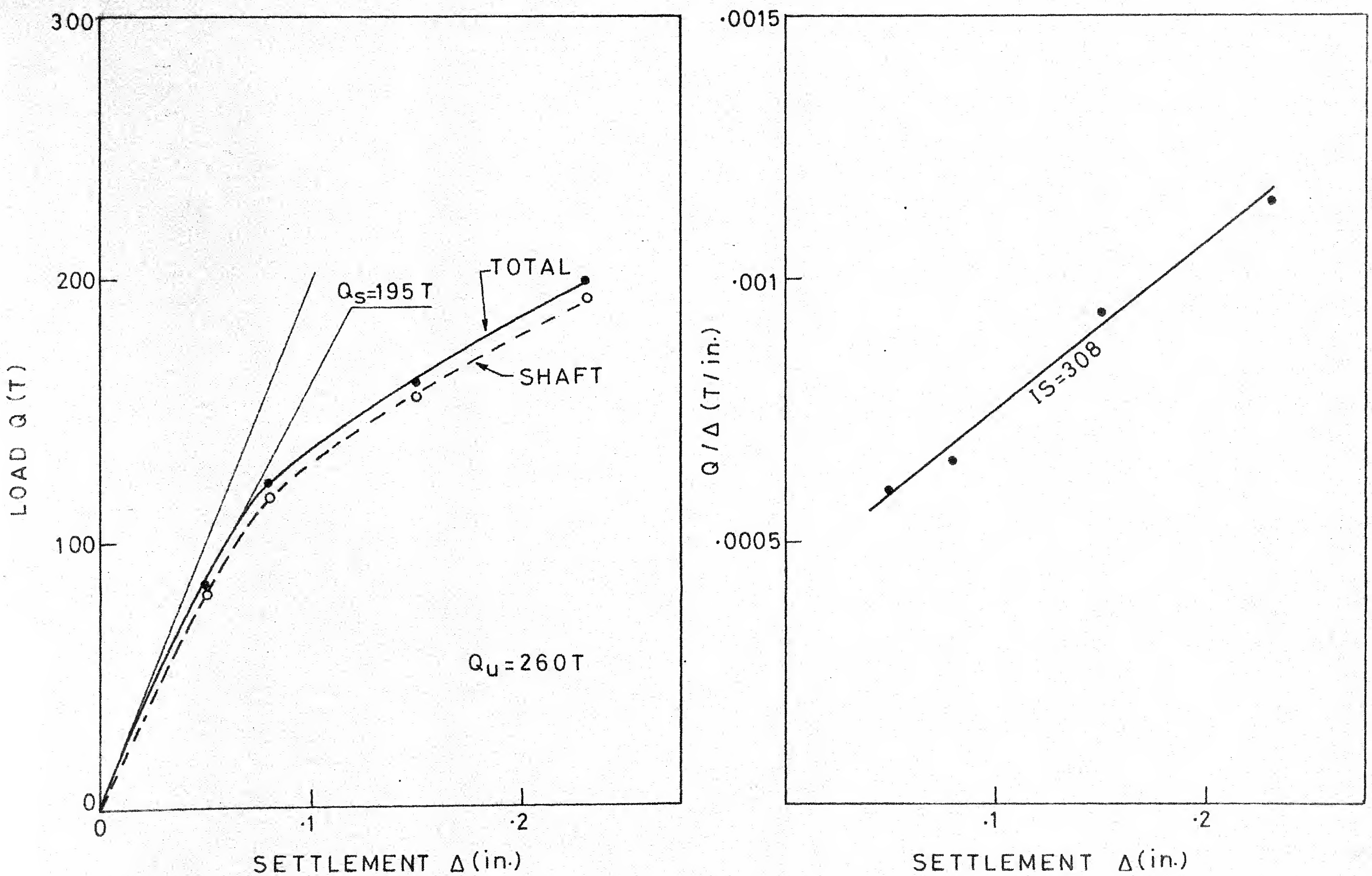


FIG. 4·3 LOAD-DISPLACEMENT CURVE FOR PILE K

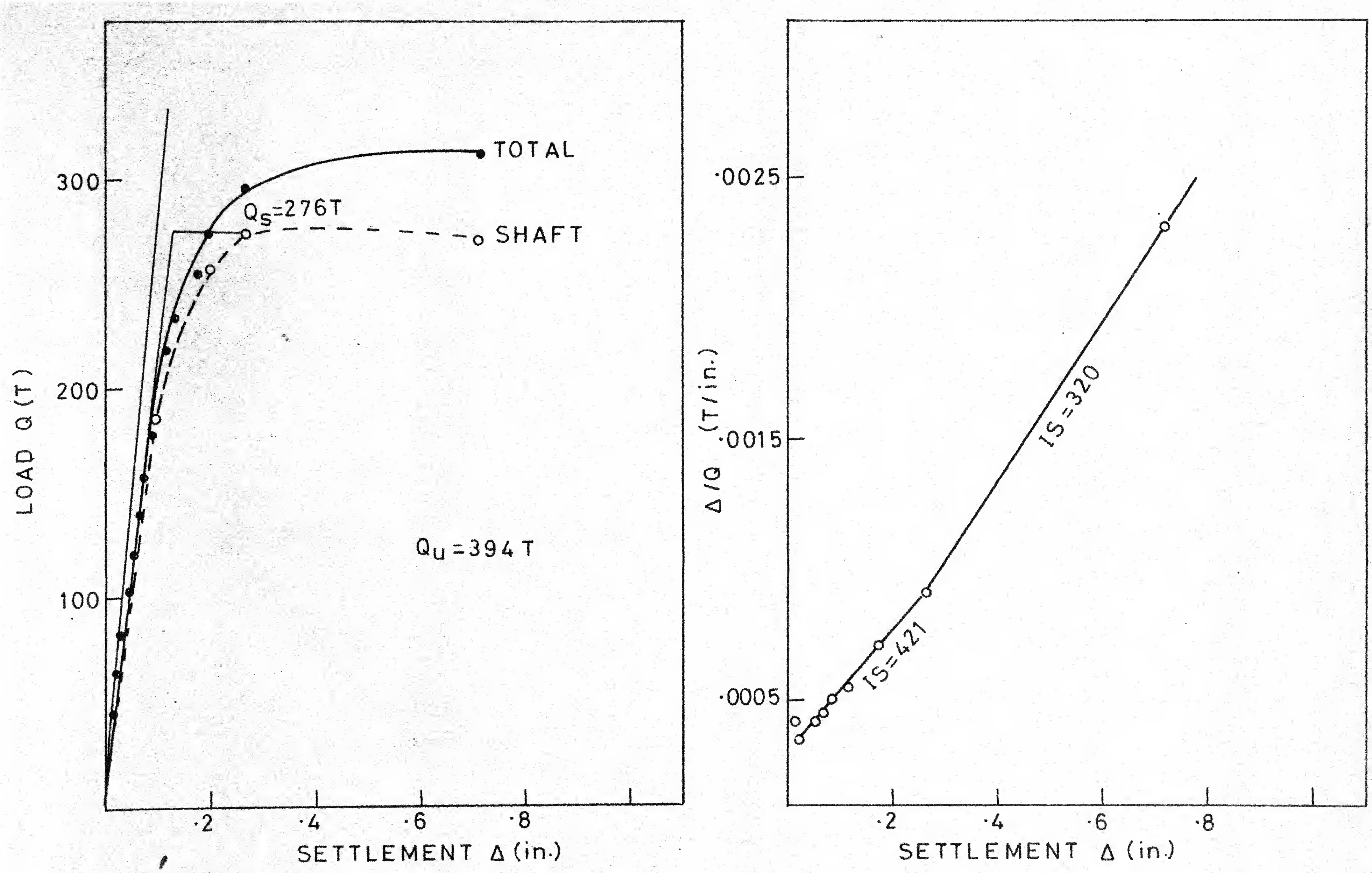


FIG. 4·4 LOAD-DISPLACEMENT CURVE FOR PILE N

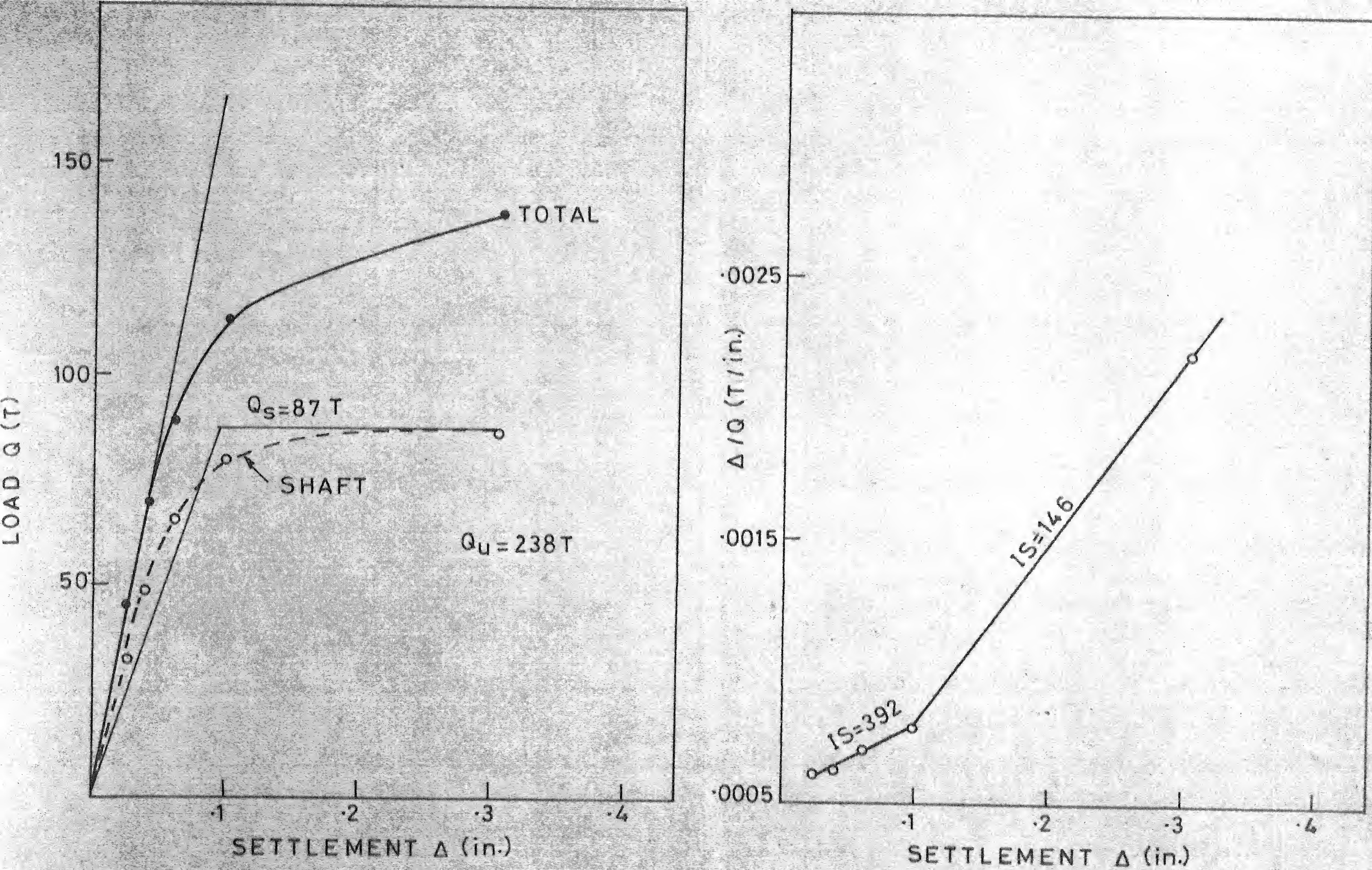


FIG. 4.5 LOAD-DISPLACEMENT CURVE FOR PILE E

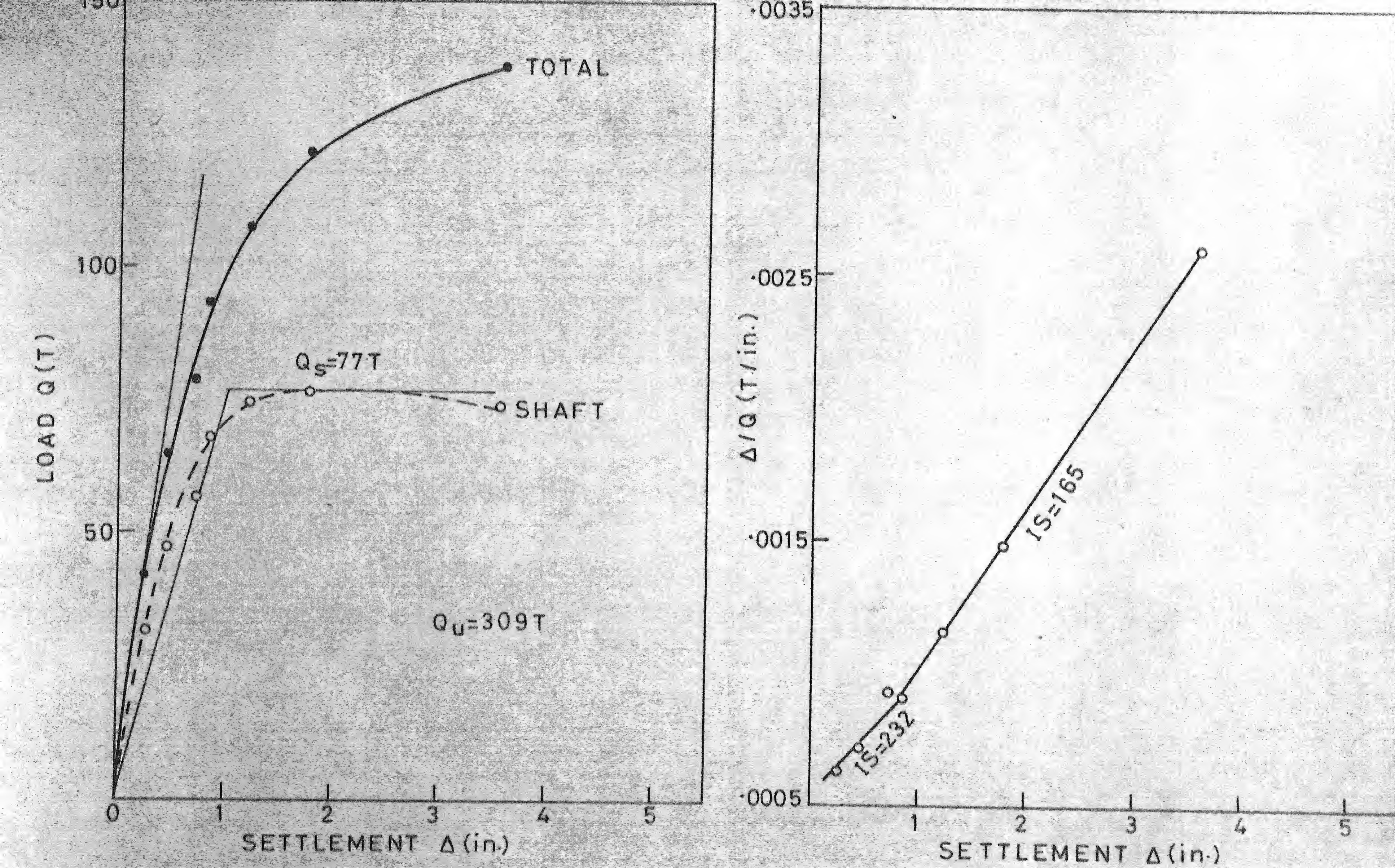


FIG. 4·6 LOAD–DISPLACEMENT CURVE FOR PILE L

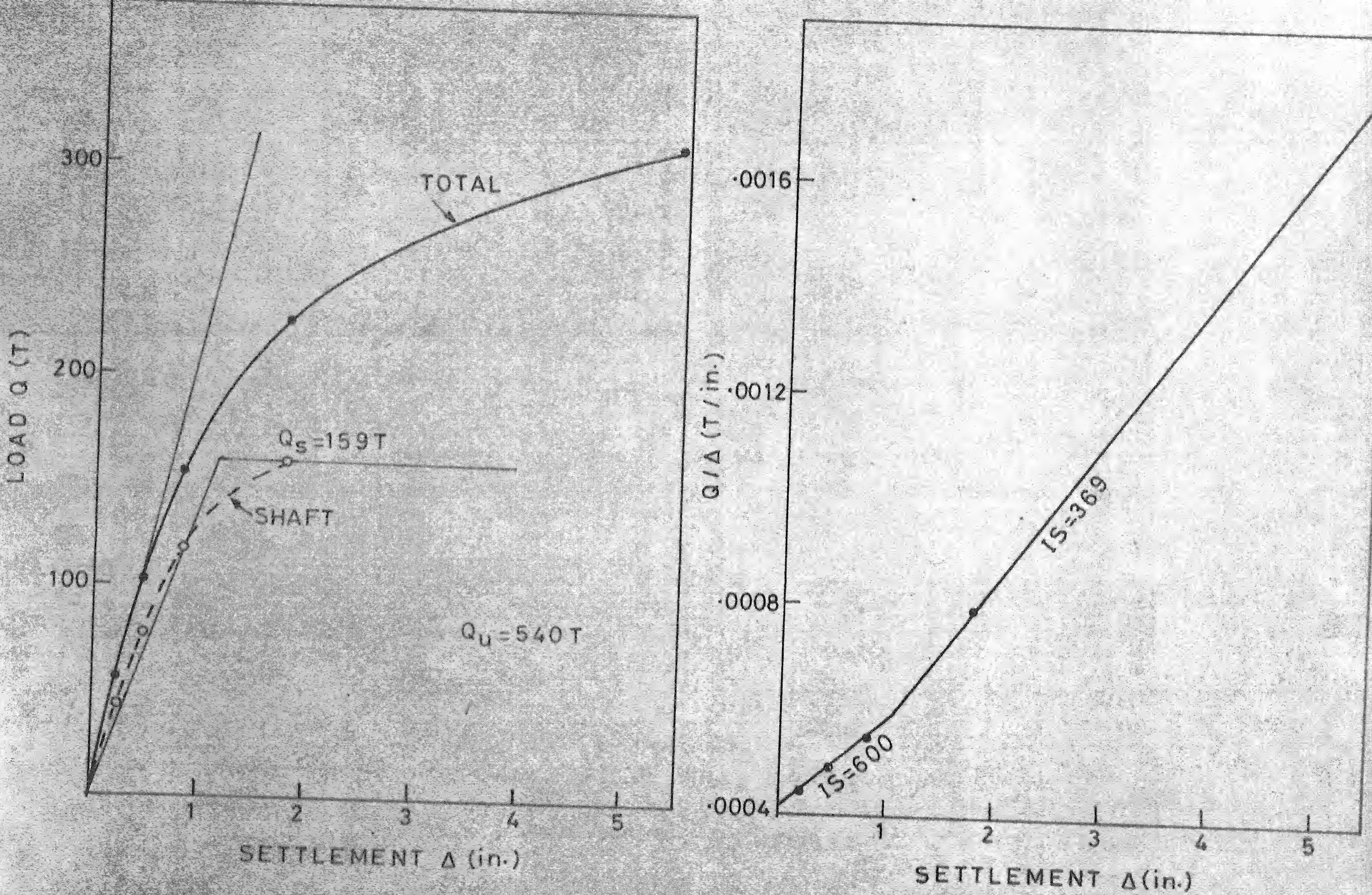


FIG. 4.7 LOAD-DISPLACEMENT CURVE FOR PILE M

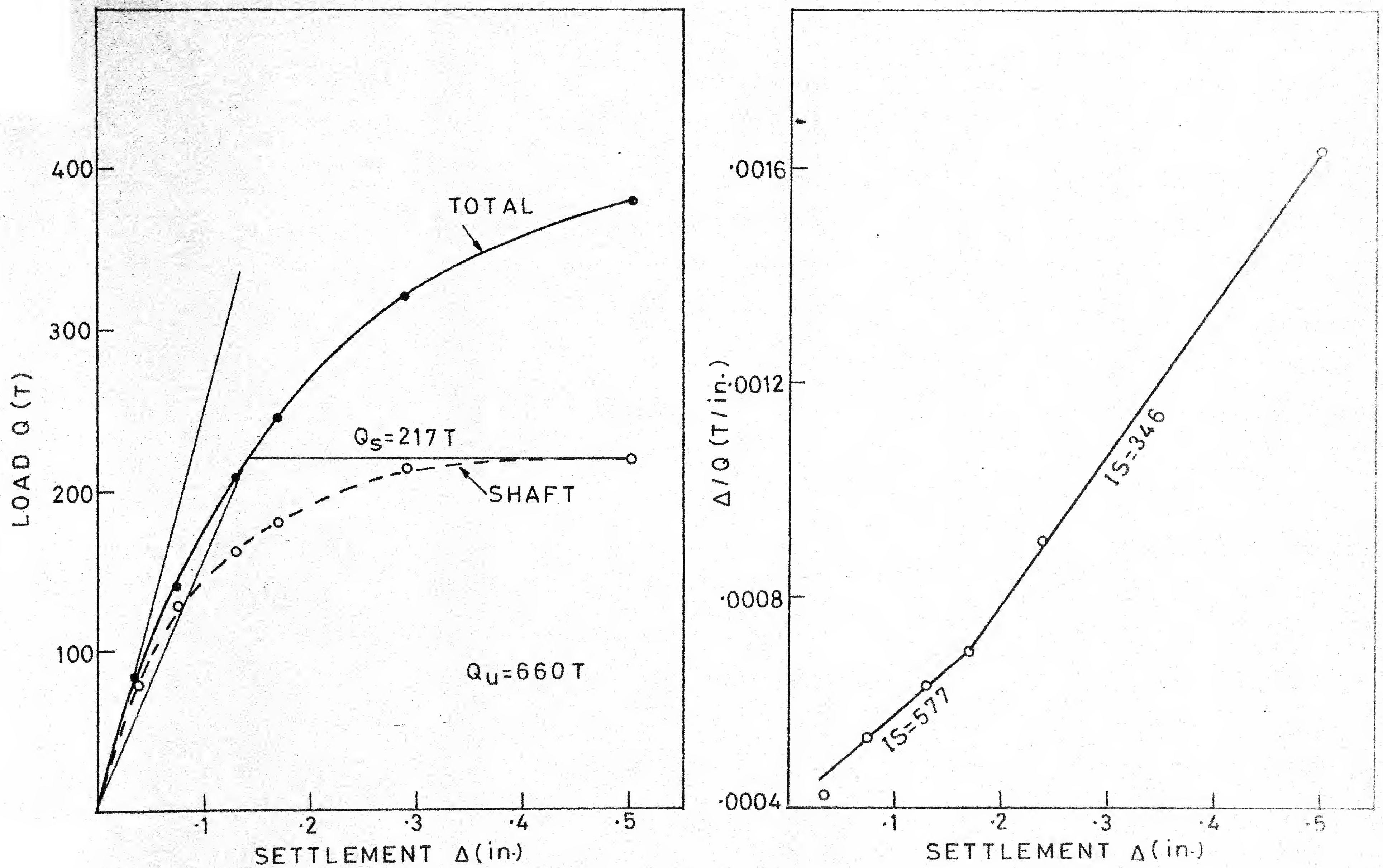


FIG. 4·8 LOAD-DISPLACEMENT CURVE FOR PILE P

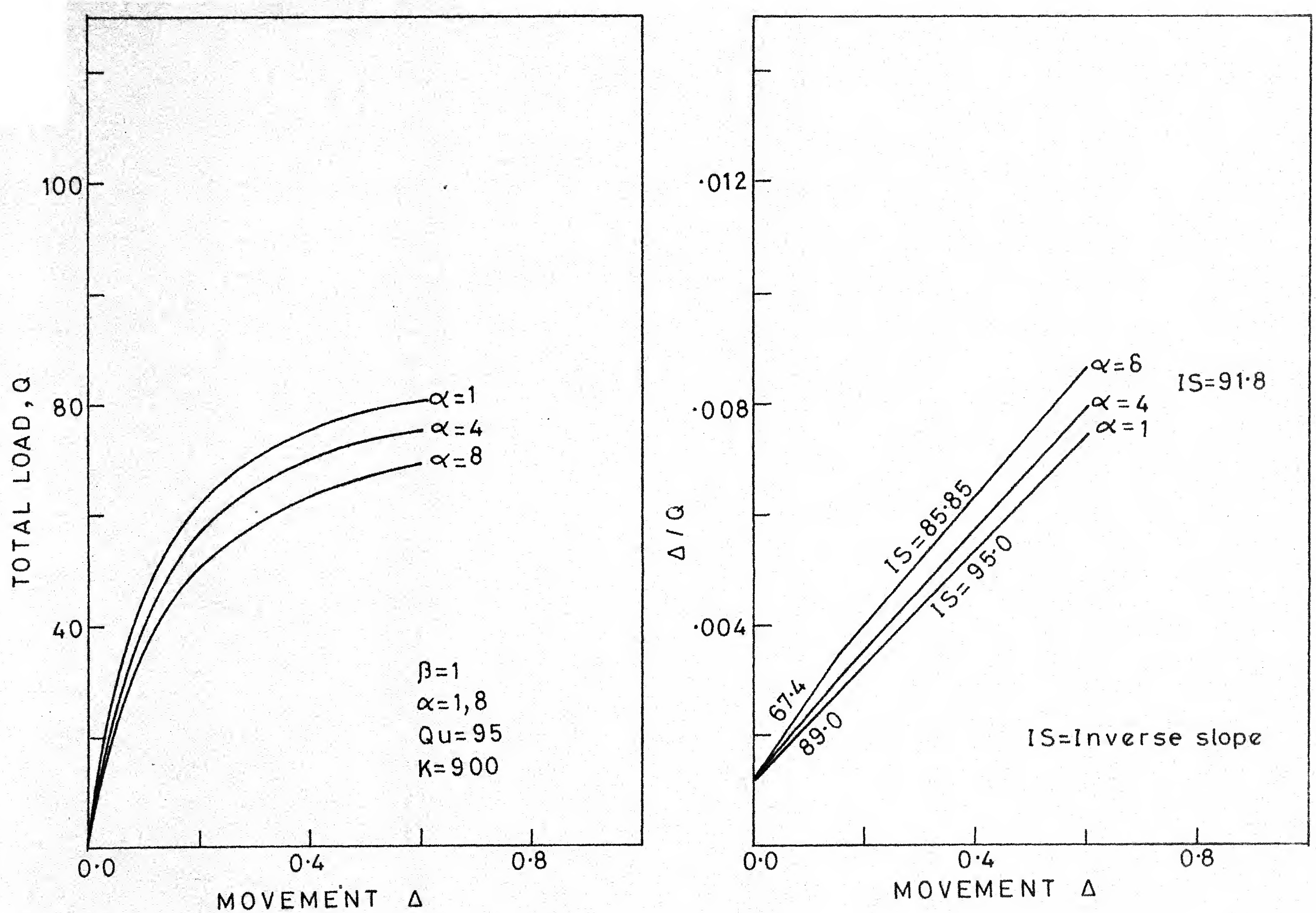
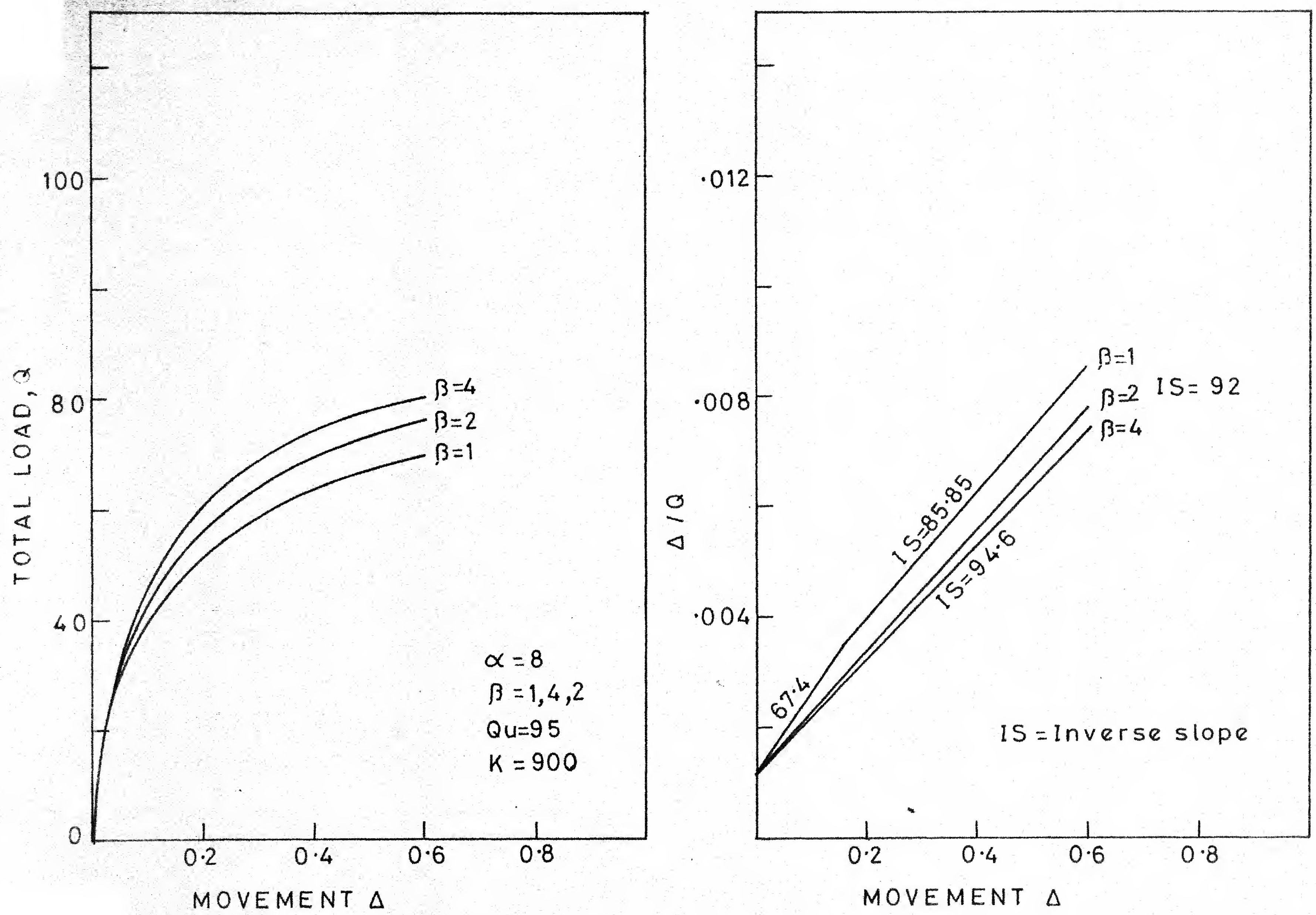


FIG. 4·9 THEORETICAL LOAD-SETTLEMENT CURVES
(β-CONSTANT)



FIG·4·10 THEORETICAL LOAD-SETTLEMENT CURVES

(α-CONSTANT)

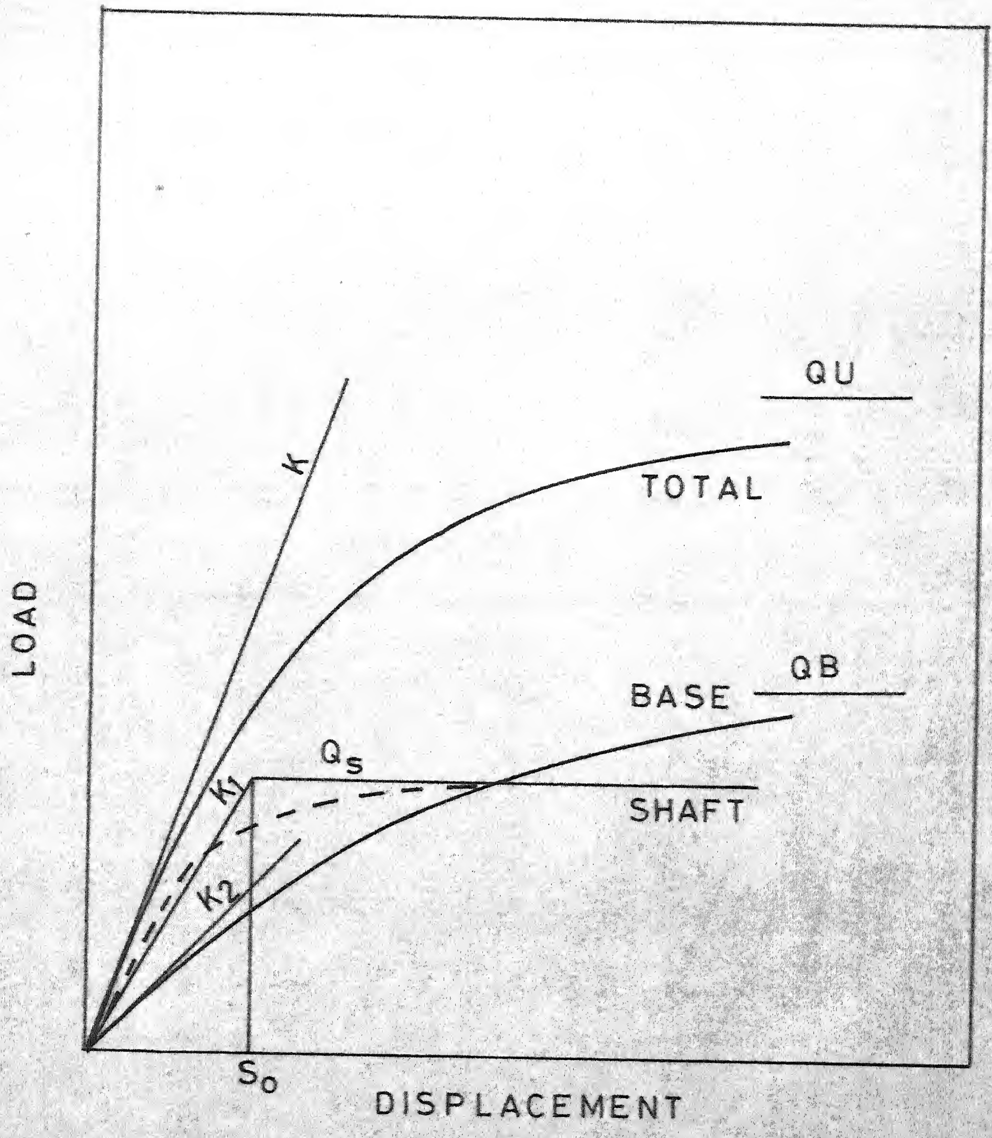


FIG. 4.11 LOAD-DISPLACEMENT CURVES ASSUMED IN PROPOSED METHOD

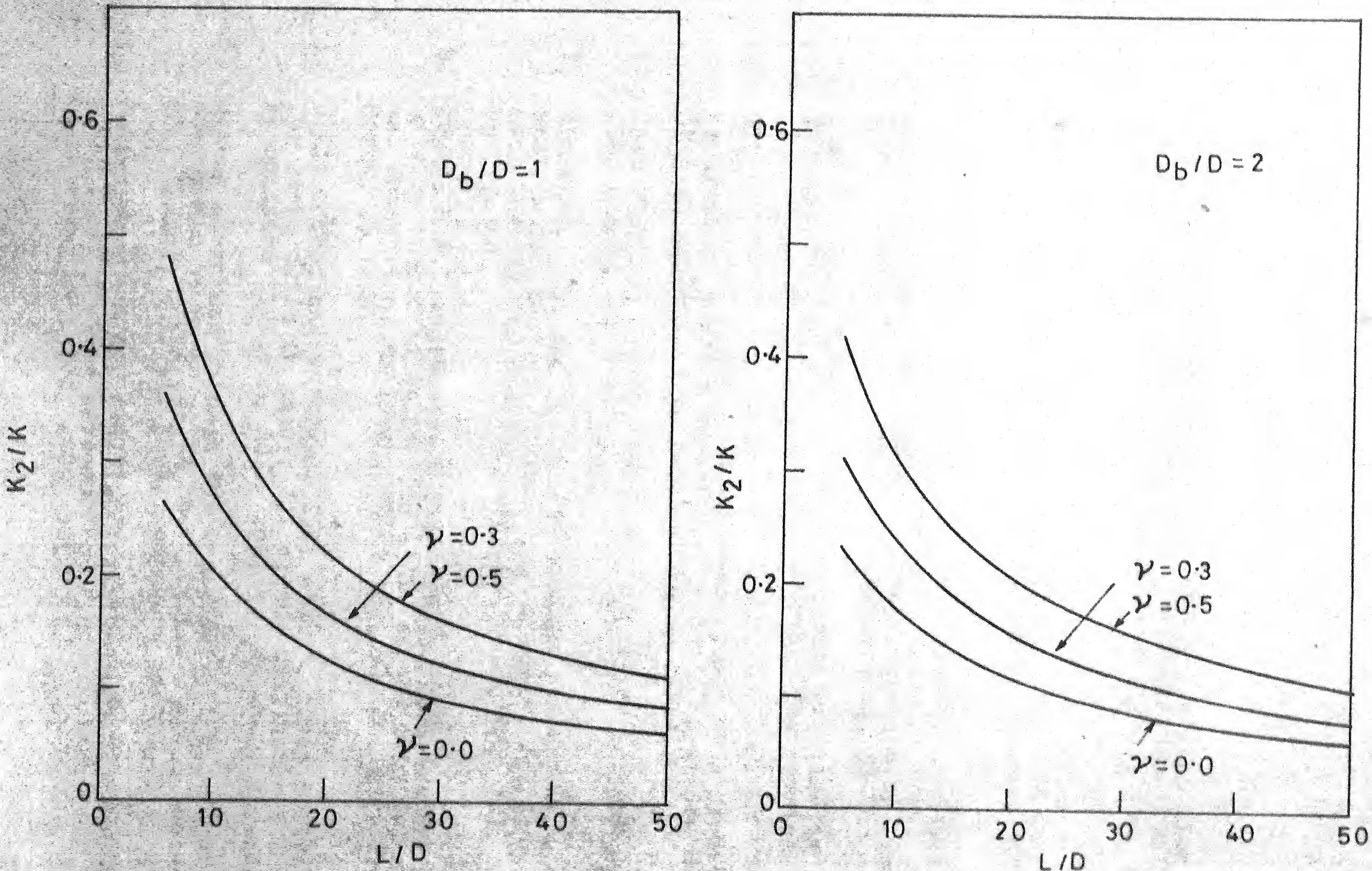


FIG. 4·12 VALUES OF K_2/K AS FUNCTION OF L/D D_b/D AND ν

REFERENCES

1. Bazant, Z., and Masopust, J. (1980), 'Drilled pier design based on load-settlement curve', Proceedings 10th International Conference on SM and FE, Rotterdam, vol. 2, pp 615-618.

2. Bjerrum, L. (1973): 'Problems of soil mechanics and construction on soft clays', 8th International Conference on Soil Mechanics and Foundation Engrg. Moscow, vol.3, pp. 111-159.

3. Burland, J.B., Butler, F.G., and Dunican, P., (1966), 'The behaviour and design of large diameter bored piles in stiff clay', Proceedings of the Symposium on Large Bored Piles, I.C.E., London, February, pp. 51-71.

4. Burland, J.F. (1973) : 'Shaft friction of piles in clay; A simple fundamental approach', Ground Engineering, vol. 6, No.3, pp 30-32, 37-42.

5. Chin Fung Kee (1972), 'The inverse slope as a prediction of ultimate bearing capacity of piles', Proc. 3rd South East Asian Conf. Soil Engineering, pp 83-91.

6. Chin Fing Kee (1977), 'DISCUSSION', Piles in Weak Rock, ICE, London, pp 212-215.

7. Clemence, P.S. and William F. Brumund (1975), 'Large scale model test of drilled pier in sand', Jl. of the Geotechnical Engineering Division, June, vol. 101, pp 537-550.

8. Coyle, H.M., and Reese, L.C., (1966), 'Load-transfer for axially loaded piles in clay', Jl. of the Soil Mechanics and Foundations Division, ASCE, vol. 92, No. SM2, Proc. March, pp 1-26.

9. Coyle, H.M., and Sulaiman, I.H. (1967), 'Skin friction for steel piles in sand', Jl. of the Soil Mechanics and Foundations Division, ASCE, vol. 93, No.SM-6, November.

10. Dinesh Mohan and Subhash Chandra (1961), 'Frictional resistance of bored piles in expansive clays', Geotechnique, London, vol. II, No.4, December, pp 294-301.

11. Fellinius , B.H., (1975), 'Test loading of piles and a new proof testing procedure', Jl. of Geotechnical Engineering Division, ASCE, vol. 101, paper No.11551.

12. Flaate, K., and Selnes, P., 'Side friction of piles in clay', Proceedings of the 9th International Conference on Soil Mech. and Foundation Engrg. Tokyo, Japan, vol. 1, pp 517-522.

13. Gregersen, O.S., Aas, G., and Dibiagio, E., (1973), 'Load tests on friction piles in loose sand', 8th International Conf. on Soil Mech. and Foundation Engrg., Moscow, vol. 2.1, pp 109-117.

14. Gruteman, M.S. and et al. (1973), 'Determination of pile resistance by means of large scale probes and pile - foundations analysis based on allowable settlements, 8th International Conf. on SM and FE, Moscow, vol. 2.1, pp 131-136.

15. Hanwa, T.H. and Tan, R.H.S., (1973), 'The behaviour of long piles under compressive loads in sand', Canadian Geotechnical Journal, vol. 10, No.3, August, pp. 311-340.

16. Holmquist, D.V., and Hudson Matlock, (1976), 'Resistance - displacement relation ship for axially loaded piles in soft clays', 8th Annual OTC Housten, Texas, vol. I, pp 553-570.

17. Janbu, N. (1976), 'Static bearing capacity of friction piles', Proceedings of the European Conference on SM and FE, vol. 1.2, pp 479-488.

18. Kraft, M. Leland, Jr., John A. Focht, Jr. and Srinath F., Amerasinghe, (1981), 'Friction capacity of piles driven into clay', Jl. of Geotechnical Engrg. Division, ASCE, November, pp 1521-1541.

19. Mattes, N.S. and Poulos, H.G. (1971), 'Model tests on piles in clay', Proc. First Australia - New Zealand Conference on Geomechanics, Melbourne, vol. 1, pp 254-260.

20. Meyerhof, G.G. (1951), 'The ultimate bearing capacity of foundations', Geotechnique, London, 2.4, pp 301-322.

21. Meyerhof, G.G. (1976), 'Bearing capacity and settlement of pile foundations', Jl.of the Geotechnical Engineering Division, ASCE, vol. 102, No.GT3, March, pp 197-228.

22. Norlund, R.L., (1965), 'Bearing capacity of piles in cohesionless soils', Jl. of Soil Mech. and Foundations Divn., vol. 89, SM 3, pp 1-35.

23. Potyondy, J.G. (1961), 'Skin friction between various soils and construction materials', Geotechnique, London, vol. 2, No.4, Dec. pp 339-353.

24. Poulos, H.G. and Davis, E.H. (1974), Elastic solutions for soil and rock mechanics, Series in Soil Engineering, John Wiley and Sons, New York, pp 180-181.

25. Poulos, H.G. and Davis, E.H. (1980), 'Pile foundation analysis and design', Series in Geotechnical Engineering, John Wiley and Sons, New York, pp 86-91.

26. Promboon, S. and R.P. Brenner, (1980), 'Large diameter bored piles in Bangkok clay', Proc. 10th International Conference on SM and FE, Rotterdam, vol. 2, pp 815-818.

27. Robert, G. Bee (1975), 'Factors affecting the axial capacity of piles in clays', 7th Annual OTC Houstan, Texas, vol. II, pp 611-623.

28. Smith, E.A.L. (1962), 'Pile driving analysis by wave equation', Transactions, ASCE, 127, Part 1, 1141-1171.

29. Terzaghi, Karl and Peck, Ralph B., (1967), Soil Mechanics in Engineering Practice, John Wiley and Sons, New York.

30. Tomlinson, M.J. (1957) :'The adhesion of piles driven in clay soils', 4th International Conf. on Soil Mech. and Foundation Engrg., London, Proceedings, vol.2, pp 66-71.

31. Tomlinson, M.J. (1977), 'Pile design and construction practice', Viewpoint Publications.

32. Touma, F.T., Reese, L.C. (1974): 'Behaviour of bored piles in sand', Jl. of Geotechnical Engrg. divn. ASCE, vol. 100, GT7, pp 749-761.

33. Van Der Veen, C. (1957), 'The bearing of a pile predetermined by a cone penetration test', Proc. 4th International Conf. on Soil Mech. London, 2, pp 72-75.

34. Van Weele, A.F. (1957), 'A method of separating the bearing capacity of a test pile into skin friction and point resistance', Proceedings of 4th International Conference on Soil Mechanics and Foundation Engineering, London, pp. 76-80.

35. Vijayvergiya, V.N., and Focht, Jr., J.A., (1972), 'A New way to predict the capacity of piles in clay', Proceedings, IVth Annual OTC, Houston, Texas, vol. II, pp 865-874.

36. Vijayvergiya, V.N., (1977), 'Load-movement characteristics of piles', Proceedings of Ports'77 Symposium, ASCE/ Long Beach, Calif. March, pp 269-284.

37. Webb, D.L. (1977), 'The behaviour of bored piles in weathered diabase', Piles in Weak Rock, ICE, pp 63-72.

38. Whitaker, T. and Cook, W. (1961), 'A new approach to pile testing', Proc. 5th Int. Conf. Soil Mech. 2, pp 171-176.

39. Whitaker, T. and Cooke, R.W., (1965), 'Bored piles with enlarged base in London clay', Proc. 6th International Conference on Soil Mech. and Foundation Engrg., vol. 2, pp 342-346.

40. Whitaker, T., and Cooke, R.W. (1966), 'An investigation of the shaft and base resistances of large bored piles in London clay', Proc. Symp. Large Bored Piles, ICE, London, pp 7-49.

41. Whitaker, T. (1970), The design of piled foundations', Pergamon Press.

42. Williams, G.M.J., and Coleman, R.B., (1965), 'The design of piles and cylinder foundations in stiff fissured clay', Proc. of 6th International Conference on Soil Mechanics and Foundation Engrg. Montreal, vol. II, Dvns. (3-6), pp 347-351.

43. Zeewaert, L. (1960), 'Reduction of point bearing capacity of piles because of negative friction', Panamerican Conference on SMFE, 1 Mexico, Proceedings, vol.3, pp 1145-1152.